Alternatives for Stormwater Total Phosphorus Removal in the Western Avenue Project Area

[DRAFT] Final Report

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Executive Summary

It is expected that in 2010, the United States Environnmental Protection Agency (EPA) will issue a new version of the National Pollutant Discharge Elimination System (NPDES) Permit regulating municipalities with separate storm sewer systems (MS4) that discharge stormwater into the Charles River basin and tributary waterbodies. The permit will require drastic reductions of total phosphorus (TP) to reduce algae blooms caused by excessive nutrient loading. In order to estimate the necessary TP reductions, the EPA and Massachusetts Department of Environmental Protection (DEP) prepared a Total Maximum Daily Load (TMDL) Report that allocates the maximum allowable TP contribution from each community and determines the necessary TP reductions relative to the year 2000.

According to the TMDL Report and proposed conditions of the NPDES permit implementation schedule, the City of Cambridge must reduce its phosphorus contribution to the Charles River by 65.2% within ten years from permit issuance. In the report, the City was divided in two main regions: areas with combined sewers and areas with separate storm drain and sanitary sewer systems. Phosphorus loading calculations assumed that only the separated areas identified in the TMDL Report contributed phosphorus to the Charles River. Areas identified in the TMDL Report served by combined sewers were assumed to have zero phosphorus contributions, with the exception of combined sewer overflows accounted for elsewhere , because they ultimately flow to the MWRA's Deer Island Wastewater Treatment Plant. The TMDL Report considered the City's Western Avenue catchment area (which consists of the Flagg Street and Hancock Street-Western Avenue subcatchments) as a single watershed with a separated storm system, and therefore, subject to the 65.2% phosphorus reduction.

At present, only the Flagg Street subcatchment discharges into the Charles River. The Hancock-Western subcatchment is served by a combined sewer connected to the MWRA's North Charles Relief Sewer (NCRS). The City of Cambridge is currently contemplating the possibility of separating the storm and sewer systems in a portion of the Hancock-Western subcatchment because the existing combined sewer has insufficient capacity to provide adequate level of service during heavy storms. Separation of the storm and sewer systems would require an outfall to the Charles River and, consequently, an increase in the current phosphorus loading would occur. The EPA requires a 100% TP offset for newly built outfalls in order to avoid any increase in TP loads relative to the year 2000. However, since the TMDL Report considers the entire Western Ave watershed as separated, this would not constitute an increase in TP loading as its contribution was already accounted for. Thus, only a 65.2% reduction would be required for the newly separated areas. Moreover, areas left with the old combined sewer could potentially be used as future offsets as they were accounted by the EPA as a source of phosphorus.

Conversations with the EPA in June 2010 suggest that only a 65.2% TP reduction will be required in separated areas - existing and proposed – within the Western Ave watershed. EPA will confirm that 100% TP offset is not necessary after reviewing in detail the TMDL Report calculations and confirming that the Western Ave watershed was, indeed, included in the TP loading calculations. In the hypothetical case an error was found, the EPA may decide to subtract the Western Ave watershed TP contribution and recalculate the necessary TP reduction based on new loading data.

According to the Long-term National Stormwater Quality Database (NSQD) compiled by Pitt (2004), typical urban runoff for all land uses (based on over 3,000 matched sample pairs) is composed of 50% dissolved total phosphorus (TP) and 50% particulate TP. The NSQD analysis of "first flush" vs. composite runoff quality for commercial and residential land uses shows a significant presence of particulate TP as well as a significant difference between TP concentrations in samples collected during the first flush period versus those collected at a later time.

In order to confirm these findings in Cambridge, MWH collected surface runoff and pipe flow between November of 2009 and January of 2010. The objective was to confirm the existence of a phosphorus and solid particle first flush phenomenon and evaluate phosphorus distribution with respect to particle size. Surface runoff was collected in two different catch basins located in a parking lot at the intersection between Green and Pleasant Street in Cambridge, MA. The first and second runoff sampling events were performed on November 23rd (storm I) and December 9th, 2009 (storm II) respectively. Pipe flow samples were collected 3 feet downstream of a new drain manhole located in Bishop Allen Drive, near the intersection with Essex Street in Cambridge, MA. Pipe flow sampling was performed during storm I and on January 25th of 2010 (storm III).

The first flush phenomenon was confirmed for both total phosphorus (TP) and Total Solids (TS) in surface runoff. Samples collected when the very first runoff flows were observed, or shortly after, had significantly higher concentrations of both TP and TS. The distribution of TP and TS followed similar patterns which suggested most of the TP is associated to solid particulates.

The first time pipe flow samples were collected during storm I, only very small particles ($\leq$ 10 microns) were present because flow velocities in the pipe were not high enough to move larger particles. The TP was mostly dissolved phosphorus (DP). Results from storm III indicated that most of the TP (~75%) is associated to particles between 25 and 45 microns which seem to start moving at velocities around 0.70 fps. Results from this storm were not in agreement with the 50% to 50% distribution of dissolved and particle-bound phosphorus reported in the NSQD. The dissolved fraction (i.e. phosphorus in water filtered using a 0.45µm filter) was in the order of only 6 to 7%.

Based on the results, MWH evaluated different possible alternatives in order to meet the mandatory TP reduction. Different individual control practices were initially evaluated for feasibility of implementation and effectiveness: infiltration, conventional Best Management Practices (street sweeping, deep sump catch basins, and rain gardens), strategic stormwater flow deflection to sanitary sewer, Jellyfish® Systems and Sorbtive™ Filter systems.

The first alternative (infiltration) was deemed unfeasible because the soils in the area have poor hydrologic condition (mostly hydrologic soils type C and D). The project area was formerly a salt marsh in the beginning of the 20th century overlaying blue clay and subsequently became an urban land fill. Soil conditions observed by MWH during past City projects in areas adjacent to Western Avenue indicate strongly that the soils will not percolate.

Conventional BMPs were deemed to be insufficient to meet the required TP reduction. The City of Cambridge currently sweeps the area with a monthly frequency using a combination of mechanical and vacuum street sweepers. The construction of rain gardens is only feasible in a portion of the Western Ave corridor which accounts for 0.25 acres approximately (0.27% of the total 92 acres). Therefore, their implementation must be combined with other alternatives in order to achieve or get close to the phosphorus standard.

The deflection alternatives consisted of installing a spillway in the proposed (Western Ave) or existing (Flagg Street) storm drains connected to an adjacent chamber with a constant flow outlet valve discharging into the respective treatment system (MWRA, Jellyfish® or Sorbtive™ Filter). For the flow deflection and TP removal calculations, the location of such structures was assumed to be at the most downstream points of the subcatchments in order to capture the largest possible amount of flow.

In order to efficiently perform these calculations, an interactive tool was developed using Microsoft Excel. Annual deflected volume and TP reduction could then be simulated with different system configurations, with variables for spill elevation and maximum deflection flow rate. TP concentrations for different phases of the storm –dissolved, first flush and non-first-flush- were assigned based on literature values and lab results. The start and end times of the different storm phases were set based on shear stress. The first flush started when the shear stress in the conduit was large enough to move 25-micron particles and ended when a higher shear stress of $1 N/m^2$ was reached. At that point, it was considered that all particles between 25 and 45 microns, which contain most of the phosphorus, had been flushed away.

Subsequently, MWH performed a cost-benefit analysis of the different alternatives – or a combination of them – for the next 20 years of service life. The mandated TP reduction could only be achieved by deflecting the necessary stormwater volume to the MWRA system. The MWRA deflection alternative could achieve the mandatory TP reduction by itself or in combination with conventional BMPs. Since it was assumed that the City of Cambridge will implement conventional BMPs such as street sweeping regardless of the TP reduction requirements, only detail cost calculations of combined practices are included in this report. For the deflection-only alternative, only an approximation of the cost is presented as a footnote below the summary tables.

Jellyfish and Sorbtive Filter technologies combined with conventional BMPs would not achieve the 65.2% TP reduction. These options were based on installing one 2-cfs Jellyfish unit or one 1.65-cfs Sorbtive Filter unit in the most downstream point of each subcatchment. Calculations with more than one unit per subcatchment were not performed due to the City's concerns about geographic space limitations for construction, installation and operational costs, and the level of commitment necessary for maintenance.

A summary TP removal efficiencies and estimated construction costs for the individual and combined alternatives are presented below.

Table 1
Cost and Removal Efficiencies of Individual TP Removal Alternatives

Alternative	Present Value of Costs (20 years)	TP Removal (% of Total Load)
Street sweeping (monthly, dry- vacuum	$397,972	4%
Rain gardens	$2,481,218	0.18%
BMP Catch basin	$4,723,732	2%
Deflection to MWRA	$910,930	59.0%
Deflection to Jellyfish	$1,171,687	44% (optimistic)
Deflection to Sorbtive Filter	$1,213,366	47% (optimistic)

Table 2
Cost and Removal Efficiencies of Combined TP Removal Alternatives

Alternative	Present Value of Costs (20 years)	TP Removal (% of Total Load)
Conventional BMPs	$7,602,922	6.18%
Conv. BMPs + Deflection to MWRA	$8,513,552	65.2%
Conv. BMPs + Deflection to Jellyfish	$8,774,609	50.2% (optimistic)
Conv. BMPs + Deflection to Sorbtive Filter	$8,816,288	53.2% (optimistic)

Note: A 65.2% TP reduction can be achieved through increased deflection of stormwater flow to the MWRA system. Under this scenario, the 20 year present value of cost is in the order of a 10% increase to the individual Deflection to MWRA alternative cost in Table 1.

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LIST OF ACRONYMS AND ABBREVATIONS

BMP	Best Management Practices
DP	Dissolved Phosphorus
EMC	Event Mean Concentration
EPA	Environmental Protection Agency
I/I	Infiltration/Inflow
Mass DEP	Massachusetts Department of Environmental Protection
MS4	Municipal Separate Storm Sewer System
MWRA	Massachusetts Water Resources Authority
NCRS	North Charles Relief Sewer
NPDES	National Pollutant Discharge Elimination System
NSQD	National Stormwater Quality Database
TP	Total Phosphorus
TS	Total Solids
TMDL	Total Maximum Daily Load
WLA	Waste Load Allocation

Section 1
Introduction

1.1 BACKGROUND

The Charles River is a slow–moving river of approximately 80 miles that flows through 23 towns and cities in five counties in eastern Massachusetts. The lower section of the river – the Lower Charles River- has been declared as an impaired waterbody by the US.EPA. Blue-green algae blooms have been reported in low-flow conditions during summer time. These algae blooms are the consequence of greater availability of nutrients originated by human activity. Blue-green algae are of concern because they are a threat to public health and aquatic fauna since they release toxins and reduce the water-column dissolved oxygen. Moreover, they are aesthetically unpleasant, increase turbidity, and impede other designated uses of the Lower Charles River such as contact recreation.

Consequently, the Massachusetts Department of Environmental Protection (DEP) along with the United States Environmental Protection Agency (EPA) performed a Total Maximum Daily Load (TMDL) analysis that assigned maximum Waste Load Allocations (WLA) to different communities to improve water quality in the Charles River. The EPA's TMDL report identifies different phosphorus sources in the surrounding communities. One of the main phosphorus contributors is stormwater runoff collected and discharged by municipal collection systems. Pet waste, lawn fertilizers, phosphate-based detergents and decaying organic matter are just some examples of phosphorus-rich materials that can be easily flushed away by stormwater collection systems.

In order to enforce phosphorus loading reductions necessary to meet the target water quality standard of 10 mg/L of chlorophyll *a*; EPA has proposed a new NPDES Phase II MS4 General Permit which is likely to be officially finalized in 2010. The permit requires significant reductions in current phosphorus loading in order to meet the WLAs set forth in the EPA's *Nutrient TMDL Development for the Lower Charles River Basin* report.

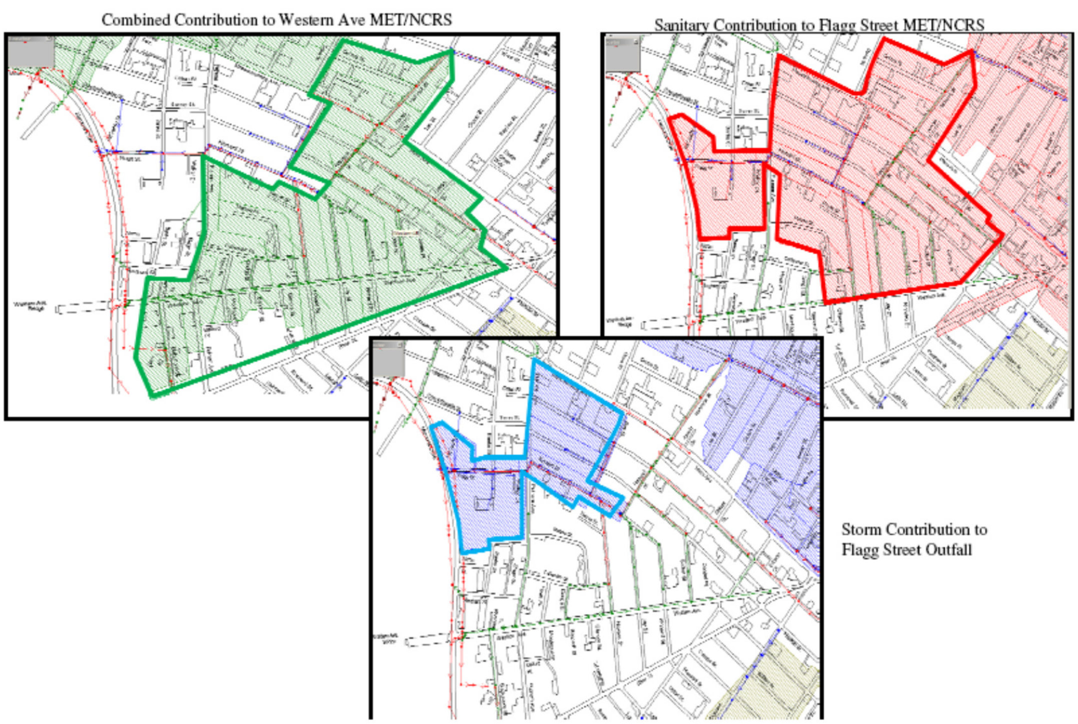
The City of Cambridge is one of the municipalities directly affected by this permit since 640 ha within the City drain to the Charles River. The required annual phosphorus reduction in the permit amounts to 65.2% of the City's estimated phosphorus annual average loading calculated between 1998 and 2002. In order to achieve such reduction, the City of Cambridge is evaluating total phosphorus (TP) removal strategies for its affected catchment areas.

1.2 PROJECT AREA

The project area (referred to as the Western Ave local drainage in the TMDL Report) is composed of two subcatchments: the Western Ave area and the Flagg Street area. The total extent is 92 acres (37.23 ha) and the dominant land uses are high density, single-family residential, commercial, and transportation (roads and sidewalks).

The Flagg Street area is currently drained by a separate storm sewer system which discharges into the Charles River through its own outfall. The Western Ave area is drained by a combined sewer system that collects sewer and storm flows discharging them into the MWRA's North Charles Relief Sewer (NCRS). At the moment, no storm water is discharged into the Charles River from this subcatchment. The following figure shows the contributing areas to the existing collection systems (sewer, combined sewer and storm drains).

Figure 1-1
Project Area with Sections Contributing to Different Collection Systems



1.3 PROBLEM DESCRIPTION

The City of Cambridge must reduce its phosphorus contribution to the Charles River by 65.2% in order to meet the water quality standards set forth in the EPA's Lower Charles River TMDL Report. The whole Western Ave drainage area (which includes both Flagg Street and Western Ave systems) is listed as one of the stormwater contributing areas that drains into the Charles River and therefore, subject to phosphorus reduction.

In the current drainage conditions, only the Flagg Street storm system discharges stormwater to the Charles River since the Western Ave subcatchment is drained by a combined sewer which discharges into the MWRA's NCRS. Therefore, the current phosphorus contribution to the Charles River from this area is zero. However, the City of Cambridge is evaluating the possibility to separate the storm and sewer systems in the Western Ave subctachment because the existing combined sewer has insufficient capacity to provide adequate, area-wide level of service during heavy storms. Separation of the storm and sewer systems would result into discharging the area's stormwater into the Charles River, which presumably would be subject to the 65.2% phosphorus reduction mandated by EPA.

1.4 OBJECTIVES

The objective of this project is to provide the City of Cambridge with cost-effective alternatives to meet the required phosphorus reduction in the Western Ave and Flagg Street subcatchments.

In addition to regularly explored alternatives such as infiltration basins and conventional best management practices (BMPs), the City chose to include the alternative of deflection of "first flush" storm drain flows to sanity sewer. According to the Long-term National Stormwater Quality Database (NSQD) compiled by Pitt (2004), typical urban runoff for all land uses (based on over 3,000 matched sample pairs) is composed of 50% dissolved total phosphorus (TP) and 50% particulate TP. The NSQD analysis of "first flush" vs. composite runoff quality for commercial and residential land uses shows a significant presence of particulate TP as well as a significant difference between TP concentrations in samples collected during the first flush period versus those collected at a later time.

In order to estimate actual TP loads in the City using the "first flush" concept, the need for a detailed analysis of phosphorus distribution with respect to time, flow and solid particle size was determined. This analysis would then form the basis for a detailed evaluation of the "first flush" TP removal strategy for comparison with other alternatives.

Section 2
Stormwater Sampling & Analysis

Multiple stormwater samples were collected in order to determine the phosphorus distribution with respect to time, flow and solid particle size. Stormwater samples were collected before (surface runoff) and after (pipe flow) entering the storm system. Sampling results and analysis are described as follows.

2.1 SURFACE RUNOFF

Sampling Sites Location: Green Street Parking Lot (2 different catch basins), Cambridge, MA

Sampling Dates: November 23rd, 2009 (Storm # I) and December 9th, 2009 (Storm # II)

2.1.1 Field and Lab Work

On the respective dates, an MWH crew collected surface runoff from two catch basins located on the west side (storm I) and the east side (storm II) corners of the parking lot in Green Street Cambridge, MA (depiction in Figure 2-1). Rainfall distributions for those storms are presented in Figure 2-2 and Figure 2-3 respectively.

During this field effort, a sample was collected when the very first runoff was observed. Subsequently, more samples were collected at intervals that ranged between 5 and 10 minutes except when runoff would stop flowing due to lack of rain. This sampling protocol was designed to confirm the existence of a first flush of TP and Total Solids (TS). Sampling times for both storms are presented in Table 2-1.

In order to assess the TP distribution with respect to particle size, samples were divided into six subsamples. Five subsamples were filtered using a 250-, 106-, 45-,or 25-micron mesh or a 10- micron filter. The remaining subsample was kept unfiltered. All prepared subsamples were sent to the lab for analysis of dissolved phosphorus (DP), TP and TS. Samples from the first storm (November 23rd, 2009) were not filtered using the 25-micron sieve because the relevance of filtering with this mesh size was realized a posteriori.

Figure 2-1
Surface Runoff Sampling Sites: Green Street Parking Lot

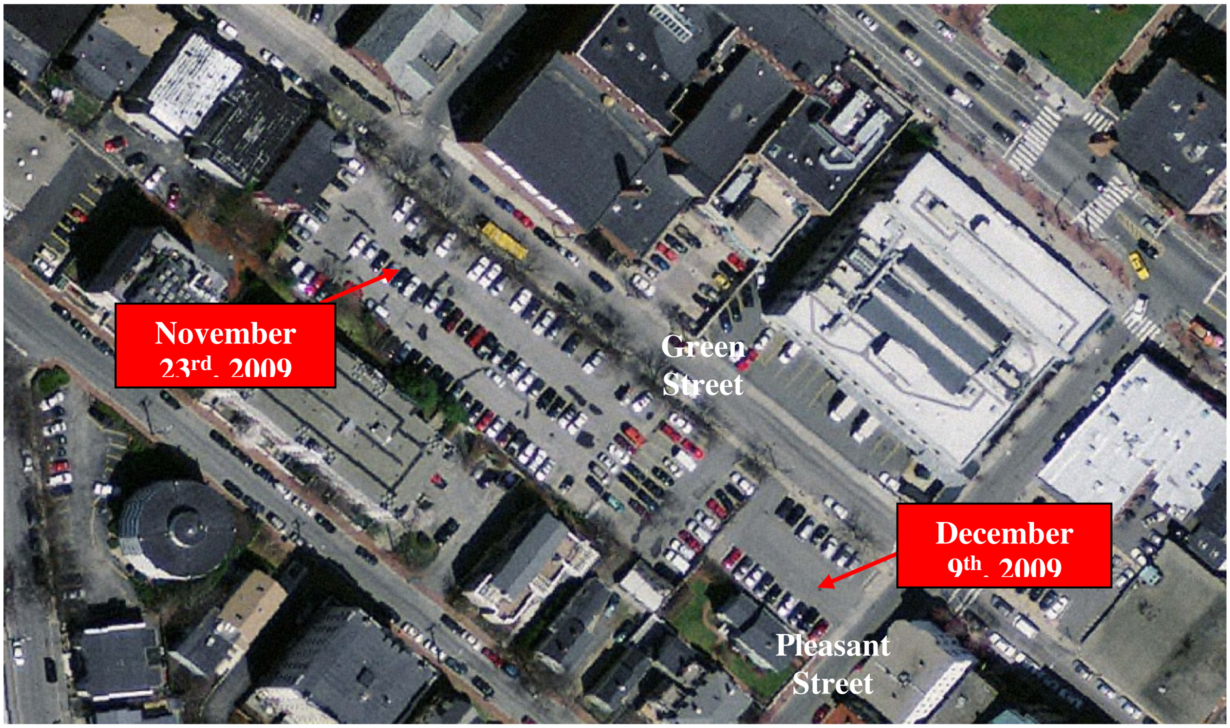


Figure 2-2
Rainfall Distribution for Storm I

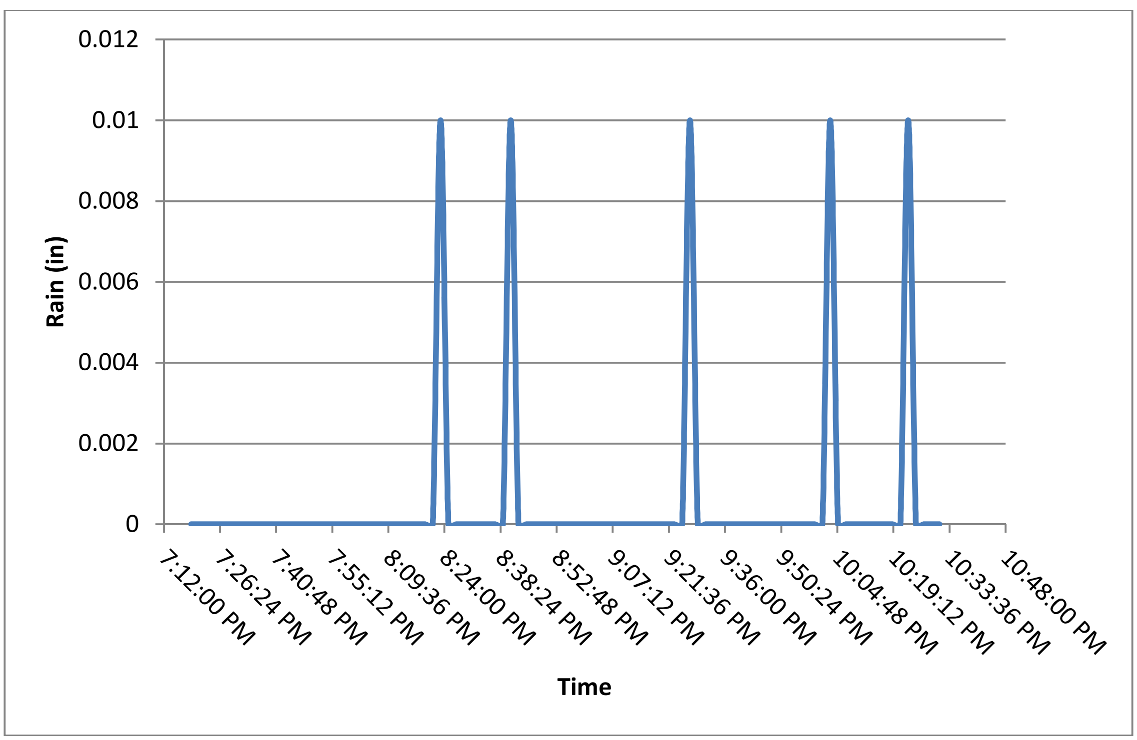


Figure 2-3
Rainfall Distribution for Storm II

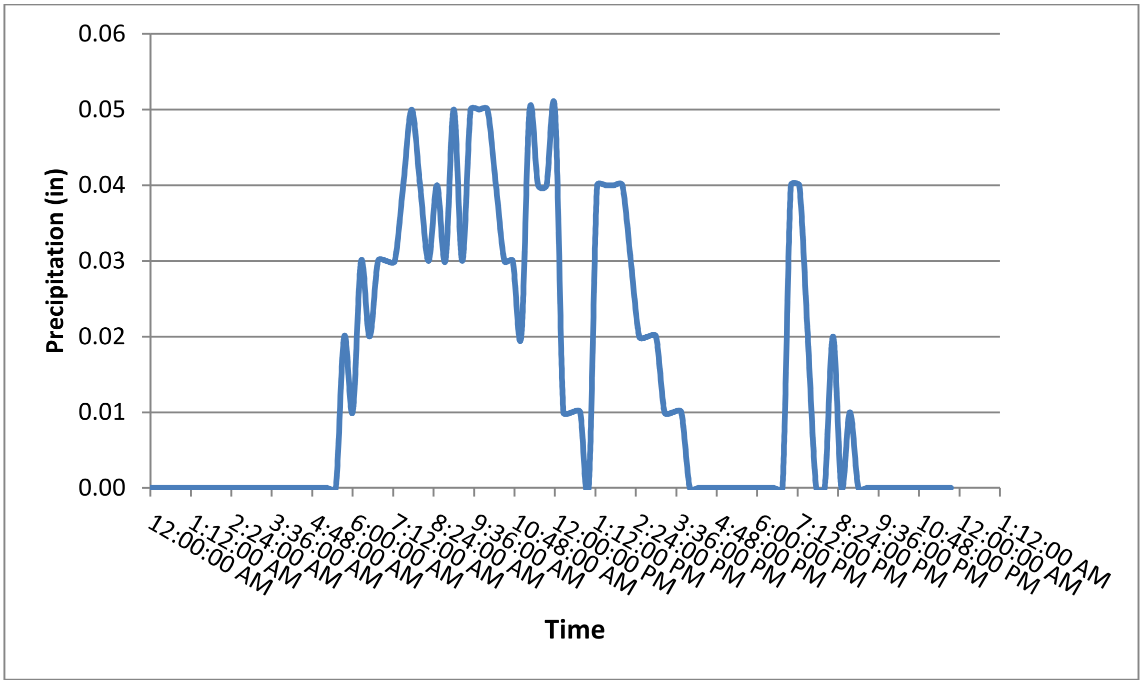


Table 2-1
Surface Runoff Sampling Times for Storm I and II

	Storm # I	Storm # II
Time Sample 1	20:30	5:35:00*
Time Sample 2	20:40	5:44:00
Time Sample 3	20:50	5:54:00
Time Sample 4	21:00	5:59:00
Time Sample 5	21:33	6:05:00
Time Sample 6	21:40	6:10:00
Time Sample 7	21:50	6:15:00
Time Sample 8	n/a	6:20:00
Time Sample 9	n/a	6:25:00

***First sample was collected a few minutes after the start of flow due to technical difficulties. Initial flow was a very small trickle into the catch basin**

2.1.2 Results

The data from the laboratory were plotted in order to visualize the evolution of TP and TS concentrations over time. Subsequently, the TP distribution with respect to particle size in the different sampling times were also calculated and plotted.

TP and TS Distribution Over Time

TP and TS distributions over time for both storms are presented in the following figures.

Figure 2-4
Surface Runoff TS Distribution for Storm I

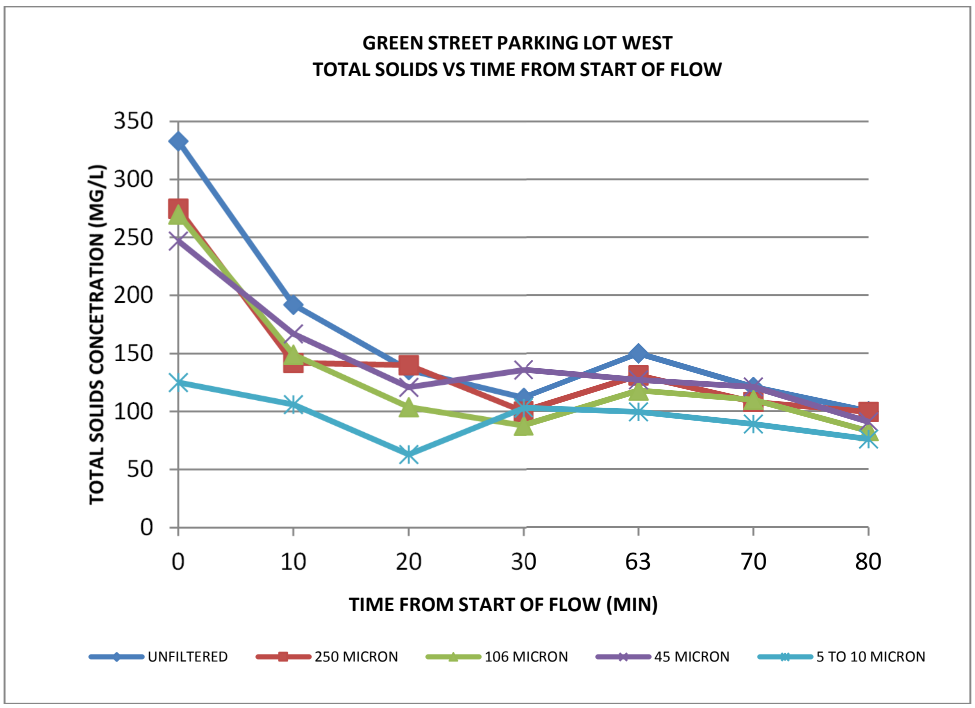


Figure 2-5
Surface Runoff TP Distribution for Storm I

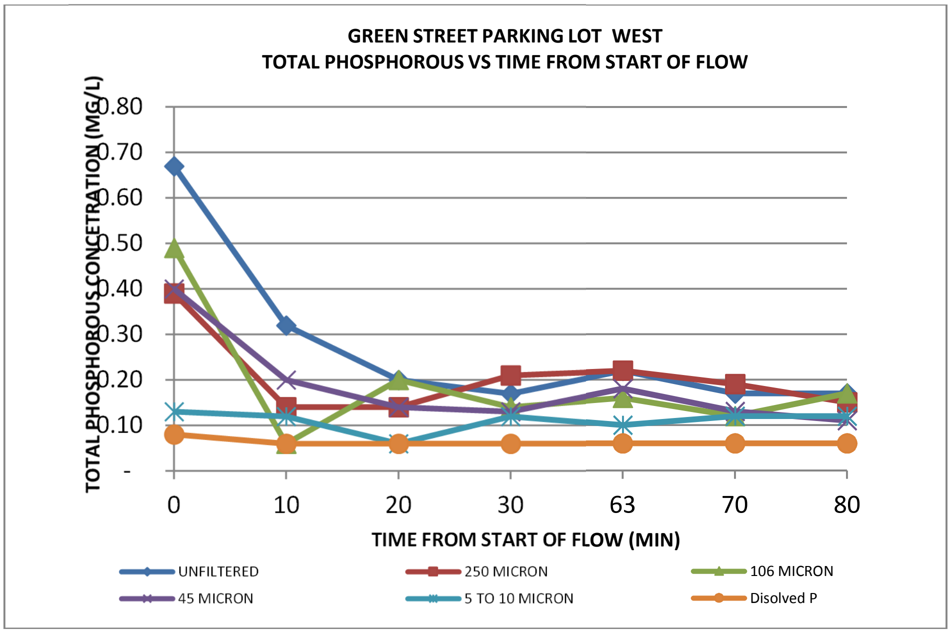


Figure 2-6
Surface Runoff TS Distribution for Storm II

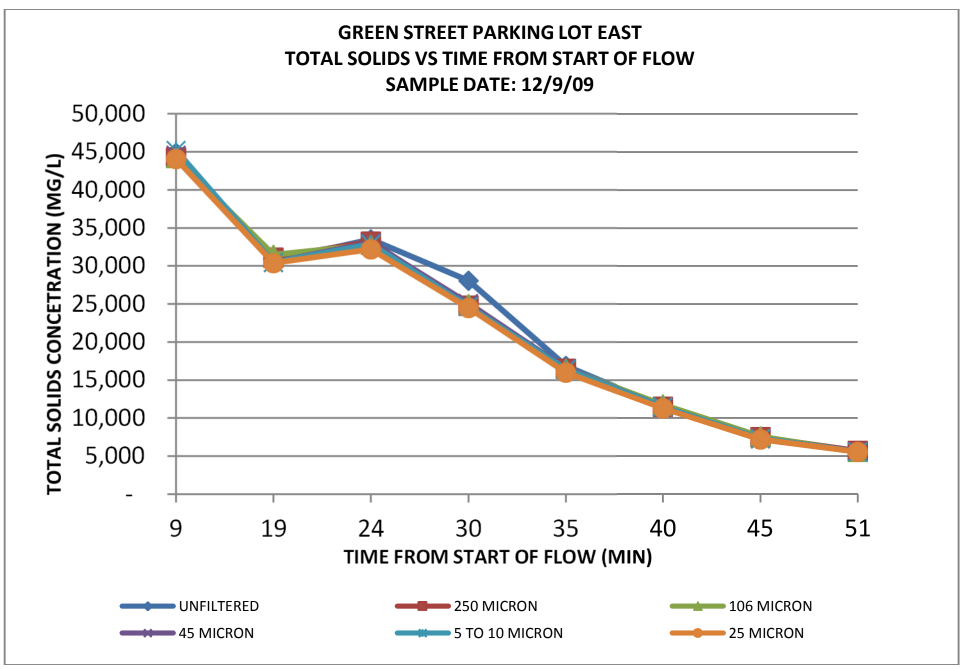
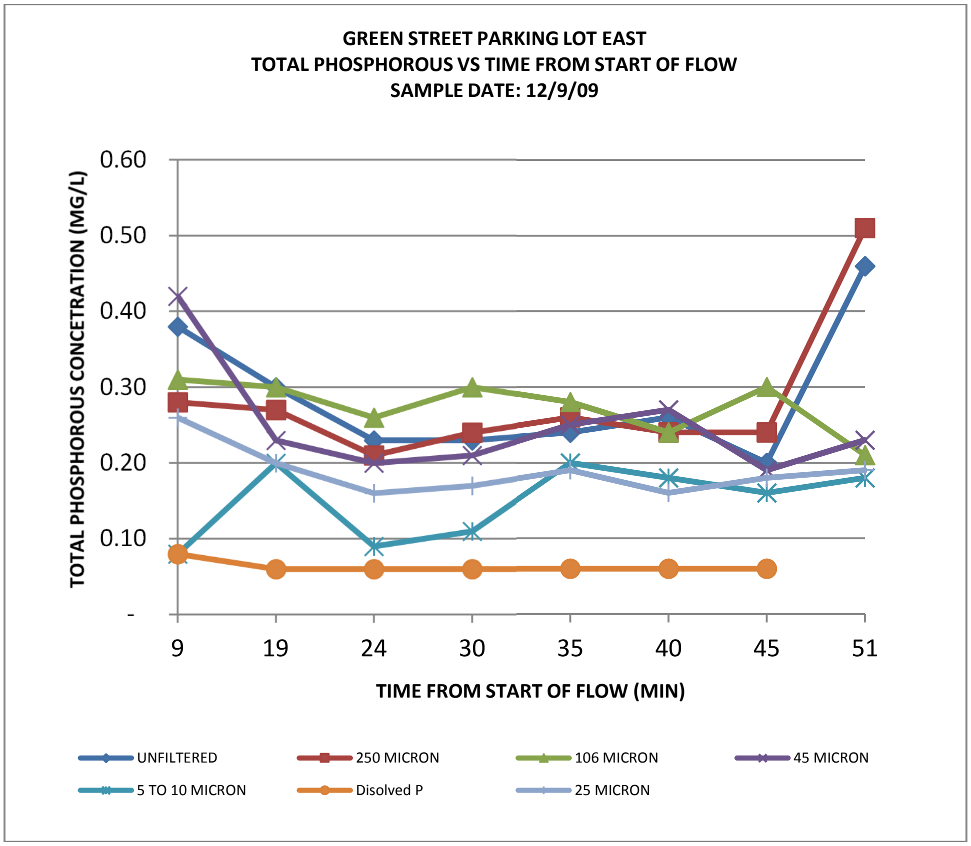


Figure 2-7
Surface Runoff TP Distribution for Storm II



TP and TS Distributions with Respect to Particle Size

The TP and TS normalized fractions associated to different particle sizes are presented in the following tables and associated figures:

Table 2-2
Normalized TP Fractions by Particle Size in Surface Runoff from Storm I

		Size Range of Solid Particulates (in microns)					
Sampling Time	Elapsed Time (min)	>250	250-106	106-45	45-10	10-0.45	DP
20:30:00	0	0.27	0.00	0.13	0.40	0.07	0.12
20:40:00	10	0.38	0.00	0.00	0.25	0.19	0.19
20:50:00	20	0.00	0.00	0.30	0.40	0.00	0.30
21:00:00	30	0.00	0.18	0.06	0.06	0.35	0.35
21:33:00	63	0.00	0.18	0.00	0.36	0.18	0.27
21:40:00	70	0.00	0.24	0.00	0.06	0.35	0.35
21:50:00	80	0.00	0.00	0.29	0.00	0.35	0.35

Figure 2-8
Normalized TP Fractions by Particle Size in Surface Runoff from Storm I

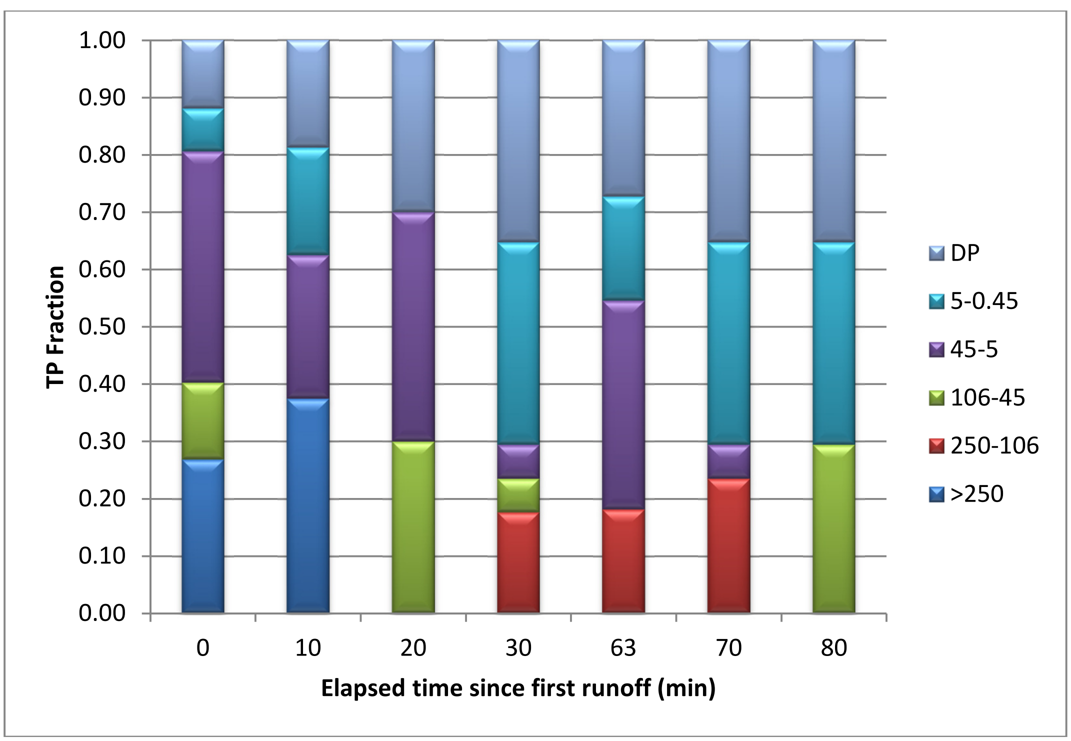


Table 2-3
Normalized TS Fractions by Particle Size in Surface Runoff from Storm I

		Size Range of Solid Particulates (in microns)				
Sampling Time	Elapsed Time (min)	>250	250-106	106-45	45-10	<10
20:30:00	0	0.17	0.02	0.07	0.37	0.38
20:40:00	10	0.13	0.00	0.00	0.32	0.55
20:50:00	20	0.00	0.11	0.00	0.43	0.46
21:00:00	30	0.00	0.00	0.00	0.08	0.92
21:33:00	63	0.13	0.03	0.00	0.18	0.66
21:40:00	70	0.00	0.00	0.00	0.26	0.74
21:50:00	80	0.01	0.09	0.00	0.15	0.76

Figure 2-9
Normalized TS Fractions by Particle Size in Surface Runoff from Storm I

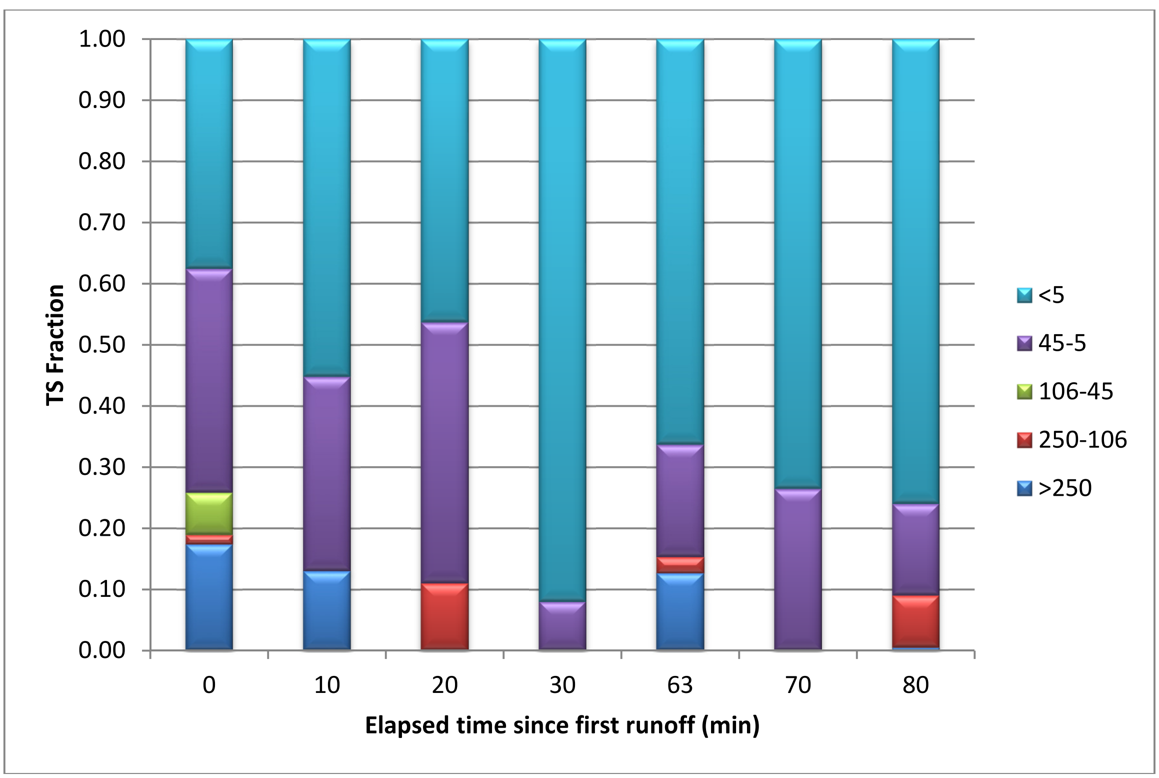


Table 2-4
Normalized TP Fractions by Particle Size in Surface Runoff from Storm II

		Size Range of Solid Particulates						
Sampl. Time	Elaps Time (min)	>250	250-106	106-45	45-25	25-5	5-0.45	DP
5:44:00	9	0.00	0.00	0.00	0.32	0.42	0.00	0.26
5:54:00	19	0.00	0.00	0.23	0.10	0.00	0.20	0.47
5:59:00	24	0.00	0.00	0.13	0.17	0.30	0.00	0.39
6:05:00	30	0.00	0.00	0.09	0.17	0.26	0.04	0.43
6:10:00	35	0.00	0.00	0.00	0.17	0.00	0.08	0.75
6:15:00	40	0.00	0.00	0.00	0.15	0.00	0.00	0.85
6:20:00	45	0.00	0.00	0.05	0.05	0.10	0.20	0.60
6:25:00	51	0.00	0.50	0.00	0.09	0.02	0.00	0.39

Figure 2-10
Normalized TP Fractions by Particle Size in Surface Runoff from Storm II

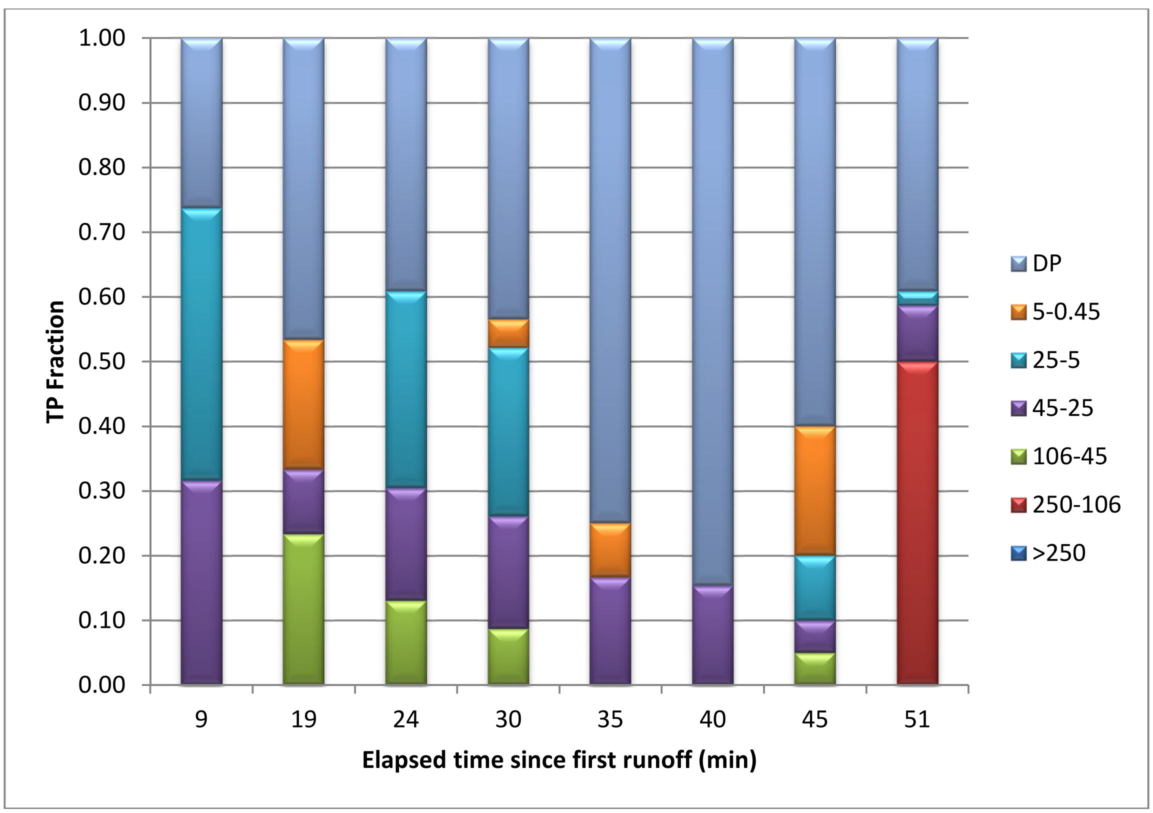
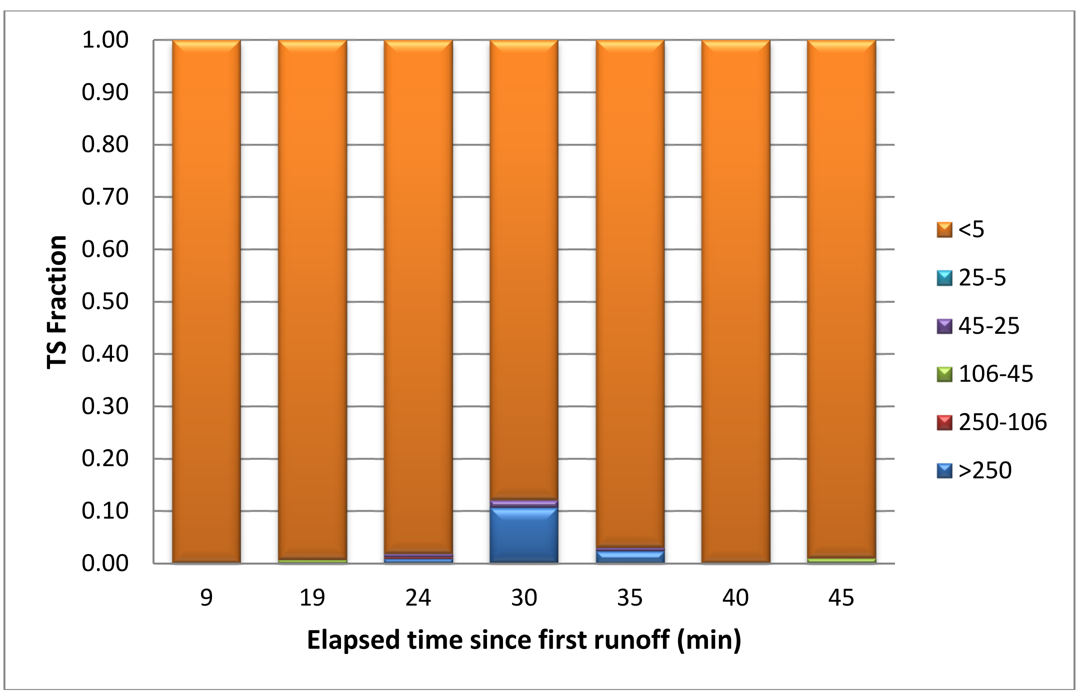


Table 2-5
Normalized TS Fractions by Particle Size in Surface Runoff from Storm II

		Size Range of Solid Particulates					
Sampling Time	Elapsed Time	>250	250-106	106-45	45-25	25-5	<5
5:44:00	9	0.00	0.00	0.00	0.00	0.00	1.00
5:54:00	19	0.00	0.00	0.01	0.00	0.00	0.99
5:59:00	24	0.01	0.00	0.00	0.01	0.00	0.98
6:05:00	30	0.11	0.00	0.00	0.01	0.00	0.88
6:10:00	35	0.02	0.00	0.00	0.01	0.00	0.97
6:15:00	40	0.00	0.00	0.00	0.00	0.00	1.00
6:20:00	45	0.00	0.00	0.01	0.00	0.00	0.99
6:25:00	51	0.00	0.00	0.00	0.00	0.00	1.00

Figure 2-11
Normalized TS Fractions by Particle Size in Surface Runoff from Storm II



2.1.3 Statistical Analysis

In order to identify statistically significant differences in TP and TS distributions across different particle size ranges, Mann-Whitney tests at a 95% confidence interval were performed. This test was selected because it is non-parametric (i.e. it doesn't make any pre-assumption on the distribution of the data). Results are shown in the following tables. Values smaller than 0.05 indicate that a statistical difference exists between pairs of data sets at the 95% confidence level.

Table 2-6
Runoff Matrix of Mann-Whittney *p* Values for Pairs of TP Distributions in Storm I

	Unfiltered	<250	<106	<45	<5	DP
Unfiltered		0.4065	0.0969	0.1416	**0.0021**	**0.0021**
<250			0.4413	0.2501	**0.0021**	**0.0021**
<106				1.0000	0.0643	**0.0100**
<45					**0.0214**	**0.0021**
<5						**0.0110**
DP						-----

Table 2-7
Runoff Matrix of Mann-Whittney *p* Values for Pairs of TS Distributions in Storm I

	Unfiltered	<250	<106	<45	<5
Unfiltered		0.4839	0.2005	0.8493	**0.0151**
<250			0.6101	1.0000	**0.0349**
<106				0.2501	0.2000
<45					**0.0214**
<5					----

Table 2-8
Runoff Matrix of Mann-Whittney *p* Values for Pairs of TP Distributions in Storm II

	Unfiltered	<250	<106	<45	<25	<5	DP
Unfiltered		0.8729	0.6746	0.2937	**0.0054**	**0.0014**	**0.0014**
<250			0.3173	0.1902	**0.0045**	**0.0009**	**0.0014**
<106				0.093	**0.0023**	**0.0009**	**0.0014**
<45					**0.0155**	**0.0028**	**0.0028**
<25						0.2713	0.0658
<5							0.8337
DP							-----

Table 2-9
Runoff Matrix of Mann-Whittney *p* Values for pairs of TS Distributions in Storm II

	Unfiltered	<250	<106	<45	<25	<5
Unfiltered		1.0000	0.9601	0.8337	0.7114	0.8337
<250			1.0000	0.9601	0.7114	0.8337
<106				1.0000	0.749	0.9601
<45					0.7114	0.9601
<25						0.7114
<5						----

2.1.4 Discussion and Conclusions

1. The existence of a first flush of TP and TS was demonstrated in storm I despite the short build-up time between consecutive rainfall events (~2.5 days) and the small amount of precipitation. The very first initial runoff was observed at 20:30 (see sampling times in Table 2-1). Samples collected at 20:30 & 20:40 had 30 to 40% of TP bound to particulates larger than 250 microns. These particulates accounted for 15% of the total TS load. In later samples, TP bound to particles 250-micron or larger disappeared but the TP associated with the next two particle gradations (250-to-106 and 106-to-45 microns) seemed to fluctuate following the hyetograph for that storm (compare Figure 2-1 with Table 2-2). The TS fractions for each size range didn't necessarily follow the TP fraction distribution. These differences are most likely due to different phosphorus binding capacities of particles of different size.

2. The first flush phenomenon was not obvious for the storm II. This could be explained by the fact that the MWH crew was not able to collect the first sample until 9 minutes after the very first runoff was observed due to technical difficulties. However, a "delayed" flush of phosphorus bound to particles between 250 and 106 microns was observed in the last sample. The collection time of this sample (6:25am, see Table 2-1) corresponds to a significant increase in rainfall intensity (Figure 2-3). The TS concentration during this storm was extremely high because of anti-icing chemicals and salt used on the street surface. Thus, the TS size distribution and its respective association to TP remains unclear for this particular storm.

3. Most of the particle-bound TP seems to be attached to particles 45 microns or smaller in size and accounted for an average 45% and 36% of the TP during the storm in November and December respectively. TP and TS distribution statistical tests between different subsamples indicated that most of the particle-bound TP is attached to these particles and returned non-significant differences between subsamples filtered with a 45 micron sieve and subsamples filtered with larger sieve openings.

4. Samples collected during storm II had an average 20% of TP bound to particles between 25 and 0.45 microns and an average 16% was bound to particles between 45 and 25 microns. These results should be taken with caution due to the elevated concentration of salt on the street which could alter the results significantly. The statistical tests indicated that significant

differences in TP concentrations did not exist between the 25-micron, 5-micron, and DP fractions but returned significant differences between the 45- and 25-micron TP fractions (see Table 2-8). This may be an indication that most of the particulate TP is contained between the 45 and 25 micron range. These results strongly agree with the statistical results from the previous storm.

5. In storm I, the average DP fraction was equal to 0.28 and its distribution was statistically different to all the particle-bound TP subsamples (Table 2-6). However, in the December storm, DP accounted for an average 52% of the TP. In this case, no significant differences were found between the DP and the small particle-bound TP distributions (25 microns and smaller). This may be due to a relatively high DP concentration compared to the TP in the 25- and 5-micron subsamples. A possible explanation for the higher concentrations of DP during this storm may be existence of a really high content of soluble particulates able to pick up larger amounts of DP on its way to the catch basin.

2.2 PIPE FLOW

Sampling Location: Drain manhole in Bishop Allen Drive, near intersection with Essex Street in Cambridge, MA

Sampling Dates: November 23rd, 2009 (Storm I) and January 25th, 2010 (Storm III)

2.2.1 Field and Lab Work

On November 23rd, 2009 and January 25 of 2010, an MWH crew collected stormwater samples from a drain manhole located in Bishop Allen Drive, near the intersection with Essex Street in Cambridge, MA (depicted Figure 2-12). An automated ISCO 6712 sampler with a 15-foot long suction line was used for this purpose. Rainfall distributions during the first and third storms are available in Figure 2-2 and Figure 2-13 respectively.

During the first storm, collection of samples started when the first flow in the 24-inch pipe was visually observed and nine more samples were collected during the following 90 minutes in 10-minute regular intervals. During the 2nd storm, sampling started when the velocity of the flow reached a set trigger value (velocity ≥ 0.75fps). A flow sensor linked to the ISCO 6712 sampler was used to determine velocity within the pipe. In this event, eleven samples were collected in 15-minute intervals during the next 2 hours and 45 minutes. Sampling times for both events are presented in Table 2-10.

Once again, samples were split in five (storm I) or six subsamples (storm III). Out of these, one subsample was kept unfiltered and the rest were filtered using 250-, 106, and 45- micron meshes and a 10-micron filter. An additional 25-micron mesh was used for the extra subsample in the January 25th storm event. TP an TS were analyzed in each subsamples. DP was analyzed in the unfiltered sample only.

Figure 2-12
Location of the Drain Manhole where Flow Samples Were Collected

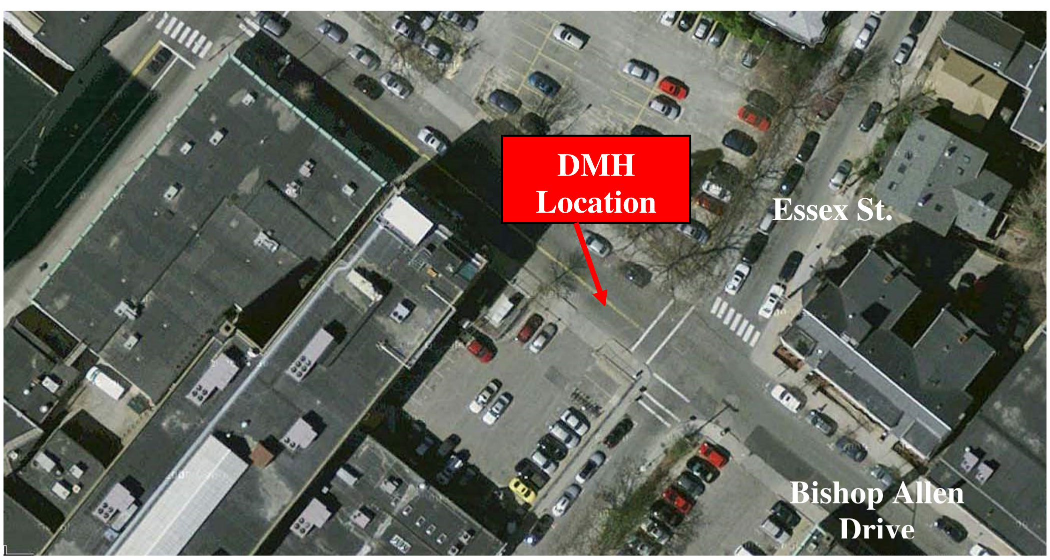


Figure 2-13
Rainfall Distribution for Storm III

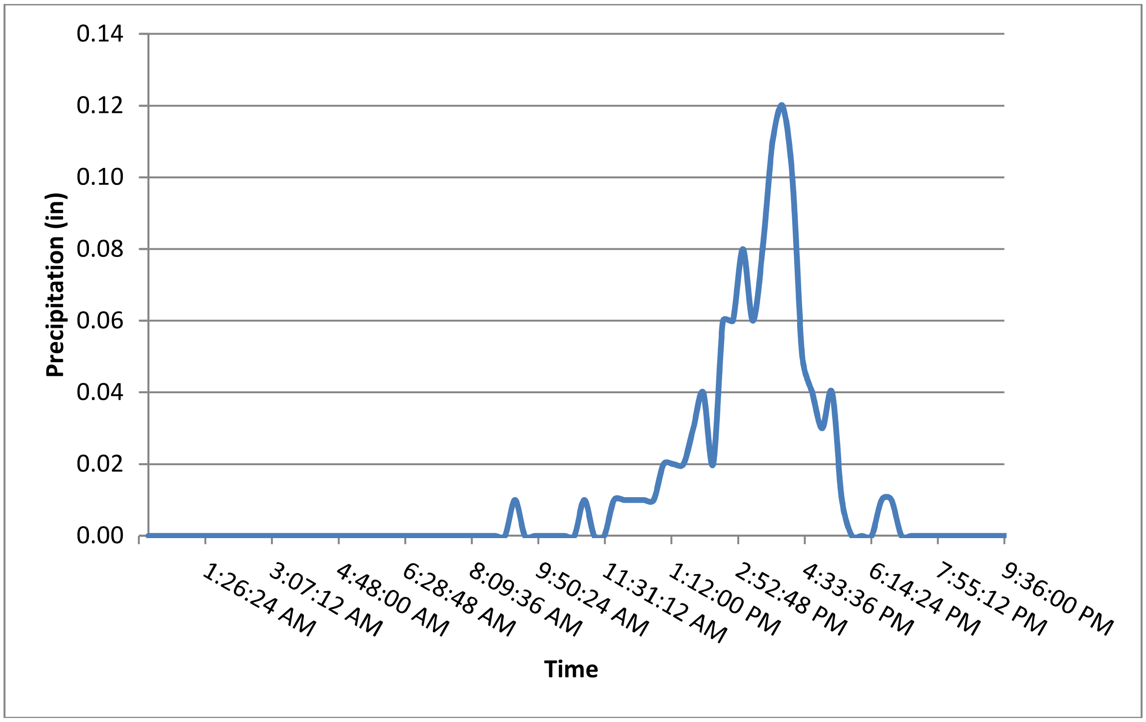


Table 2-10
Pipe Flow Sampling Times for Storms I & III

	November 23rd, 2009	January 25th, 2010
Time Sample 1	20:48	11:51
Time Sample 2	20:58	12:05
Time Sample 3	20:08	12:20
Time Sample 4	20:18	12:35
Time Sample 5	21:28	12:50
Time Sample 6	21:38	13:05
Time Sample 7	21:48	13:20
Time Sample 8	21:58	13:35
Time Sample 9	22:08	13:50
Time Sample 10	22:18	14:05
Time Sample 11	n/a	14:20
Time Sample 12	n/a	14:35

2.2.2 Results

Distributions of TP and TS with respect to time and particle size as well as recorded pipe flows were plotted. Again, statistical tests were run in order to find statistical differences in TP and TS distributions among the different particle size groups.

TP,TS and Flow Distribution Over Time

TP, TS and flow distributions during storm I and storm III are presented in the following figures.

Figure 2-14
Pipe Flow TP Distribution in Storm I

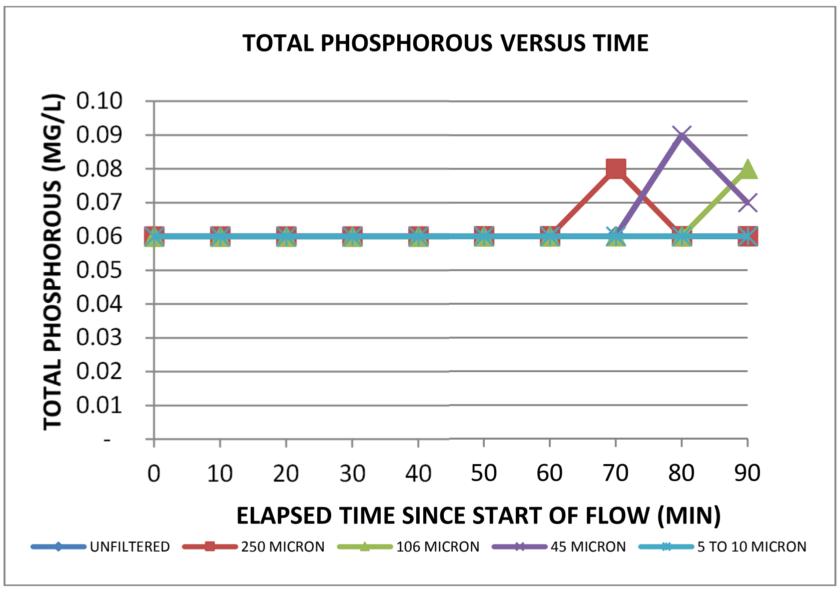


Figure 2-15
Pipe Flow TS Distribution in Storm I

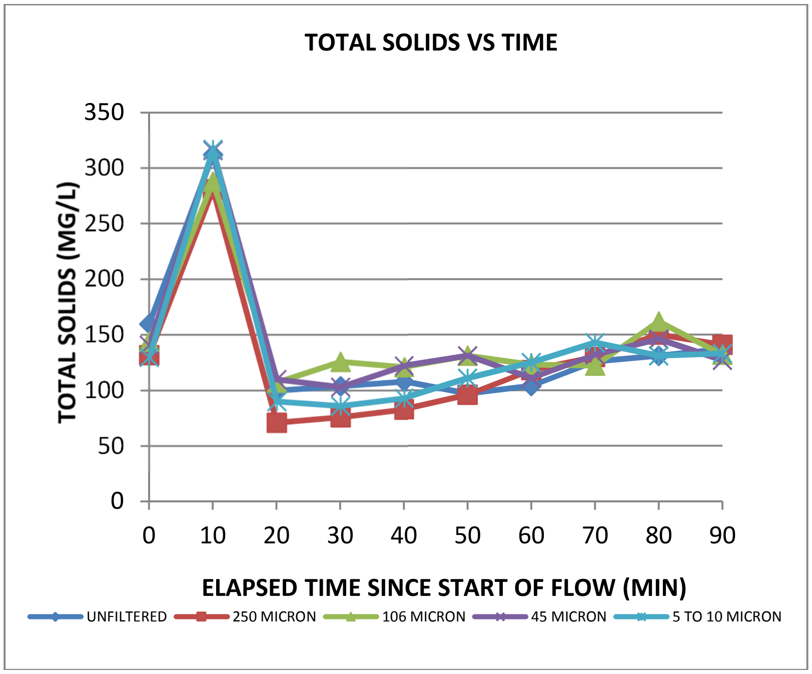


Figure 2-16
Pipe Flow and Velocity Distribution in Storm I

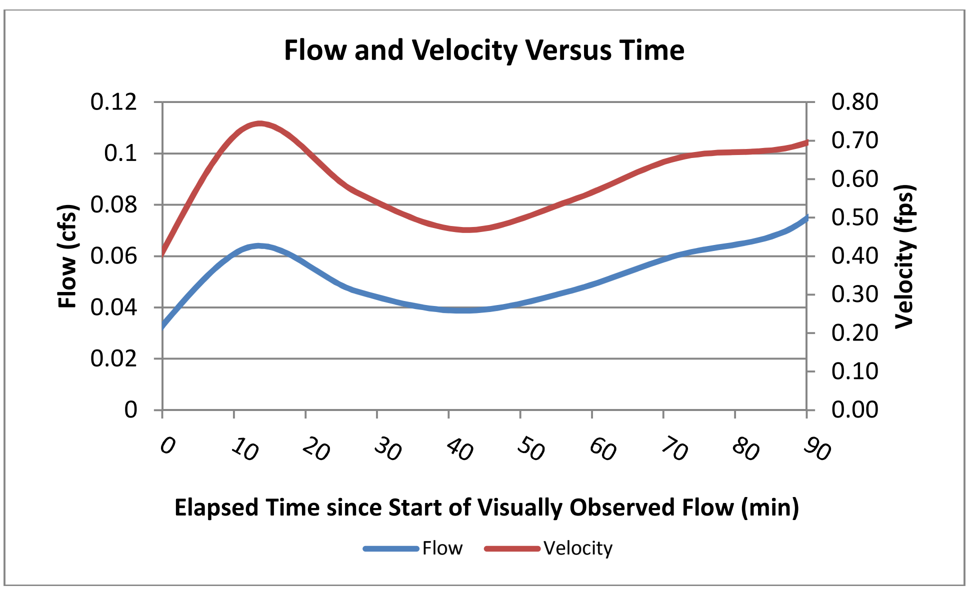


Figure 2-17
Pipe Flow TP Distribution in Storm III

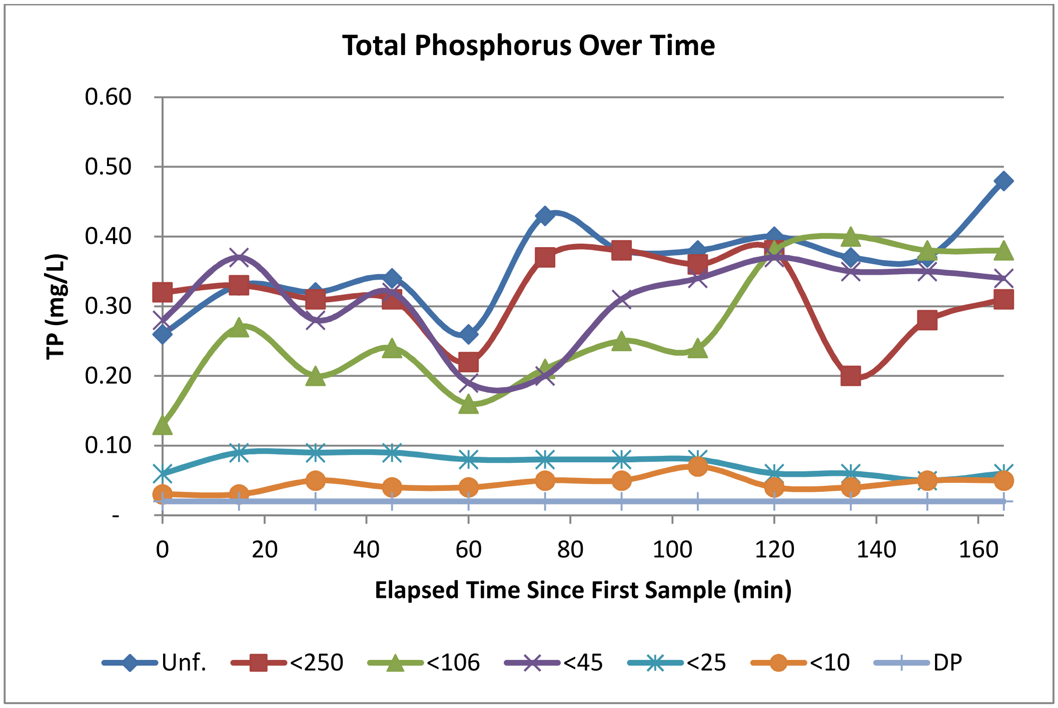


Figure 2-18
Pipe Flow TS Distribution in Storm III

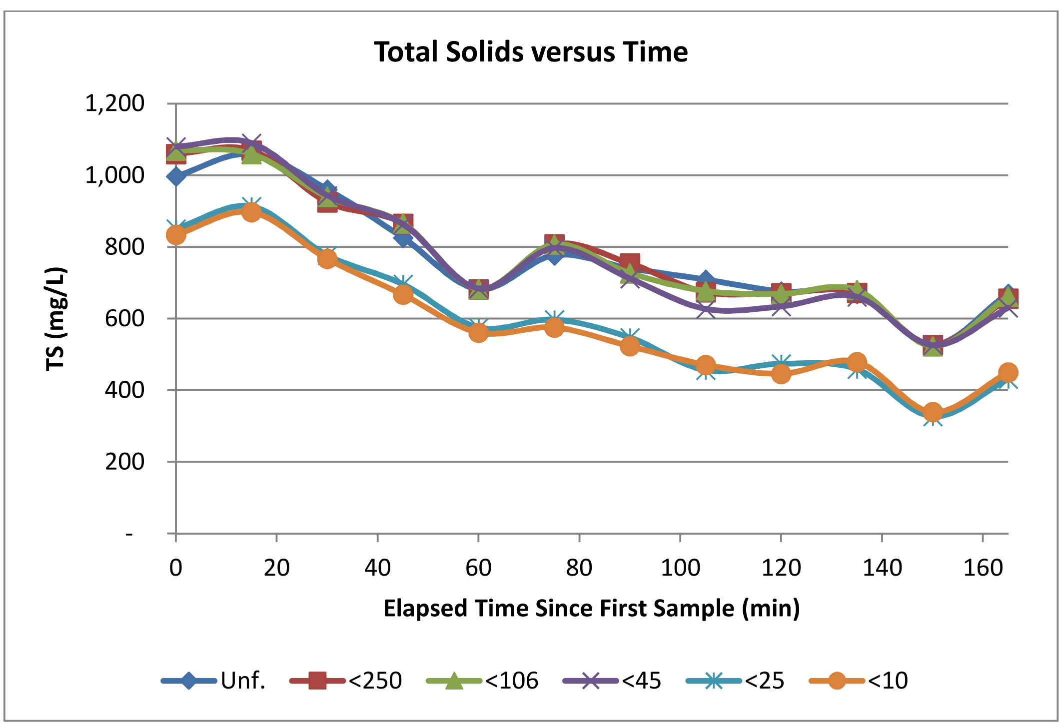
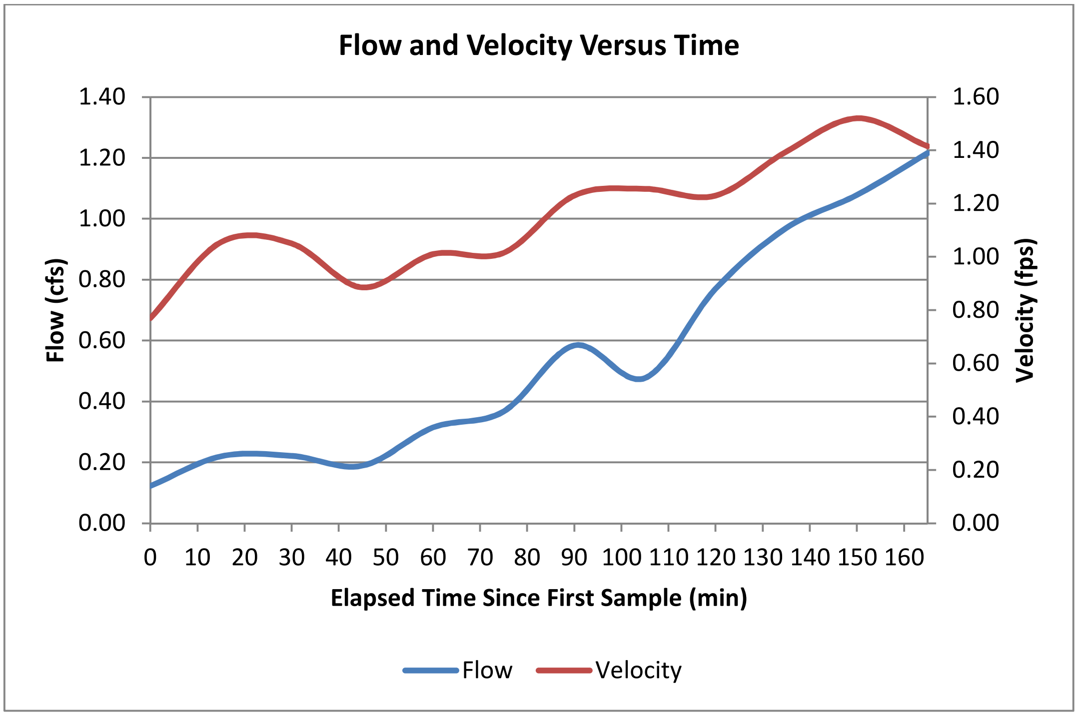


Figure 2-19
Pipe Flow and Velocity Distribution in Storm III



TP and TS Distributions with Respect to Particle Size

TP and TS normalized fractions for different particle size ranges are presented in the following tables and figures:

Table 2-11
Normalized TP Fractions by Particle Size in Pipe Flow from Storm I

		Size Range of Solid Particulates					
Time	**Elapsed Time (min)**	**>250**	**250-106**	**106-45**	**45-10**	**10-0.45**	**DP**
20:48	0	0.00	0.00	0.00	0.00	0.00	1.00
20:58	10	0.00	0.00	0.00	0.00	0.00	1.00
21:08	20	0.00	0.00	0.00	0.00	0.00	1.00
21:18	30	0.00	0.00	0.00	0.00	0.00	1.00
21:28	40	0.00	0.00	0.00	0.00	0.00	1.00
21:38	50	0.00	0.00	0.00	0.00	0.00	1.00
21:48	60	0.00	0.00	0.00	0.00	0.00	1.00
21:58	70	0.00	0.00	0.00	0.00	0.00	1.00
22:08	80	0.00	0.00	0.00	0.00	0.00	1.00
22:18	90	0.00	0.00	0.00	0.00	0.00	1.00

Figure 2-20
Normalized TP Fractions by Particle Size in Pipe Flow from Storm I

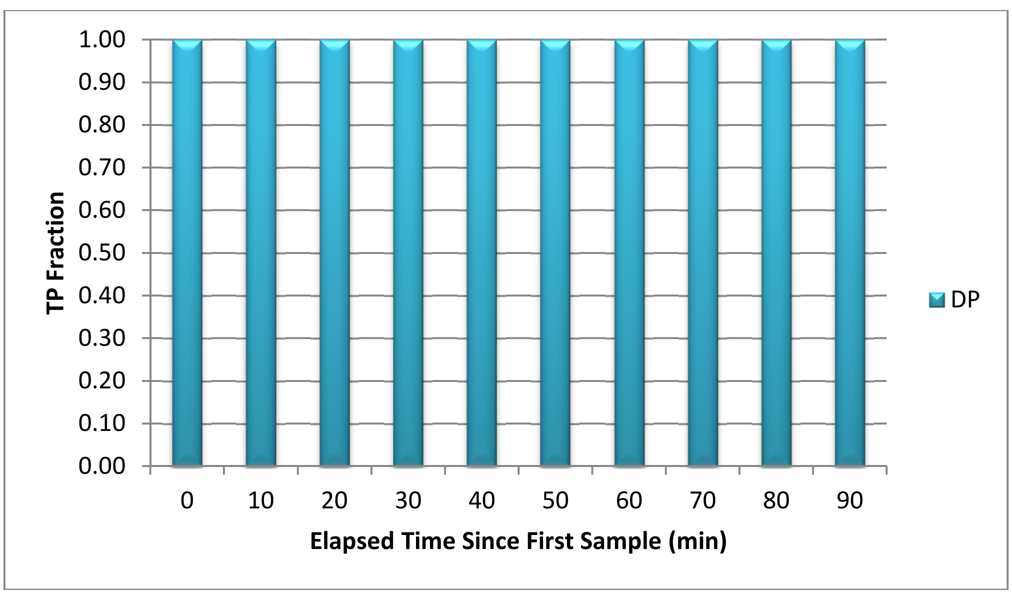


Table 2-12
Normalized TS Fractions by Particle Size in Pipe Flow from Storm I

		Size Range of Solid Particulates				
Time	**Elapsed Time (min)**	**>250**	**250-106**	**106-45**	**45-10**	**<10**
20:48	0	0.10	0.00	0.02	0.07	0.81
20:58	10	0.00	0.00	0.00	0.00	1.00
21:08	20	0.00	0.00	0.00	0.10	0.90
21:18	30	0.00	0.00	0.01	0.16	0.83
21:28	40	0.00	0.00	0.00	0.14	0.86
21:38	50	0.00	0.00	0.00	0.00	1.00
21:48	60	0.00	0.00	0.00	0.00	1.00
21:58	70	0.00	0.00	0.00	0.00	1.00
22:08	80	0.00	0.00	0.00	0.00	1.00
22:18	90	0.00	0.03	0.00	0.00	0.97

Figure 2-21
Normalized TS Fractions by Particle Size in Pipe Flow from Storm I

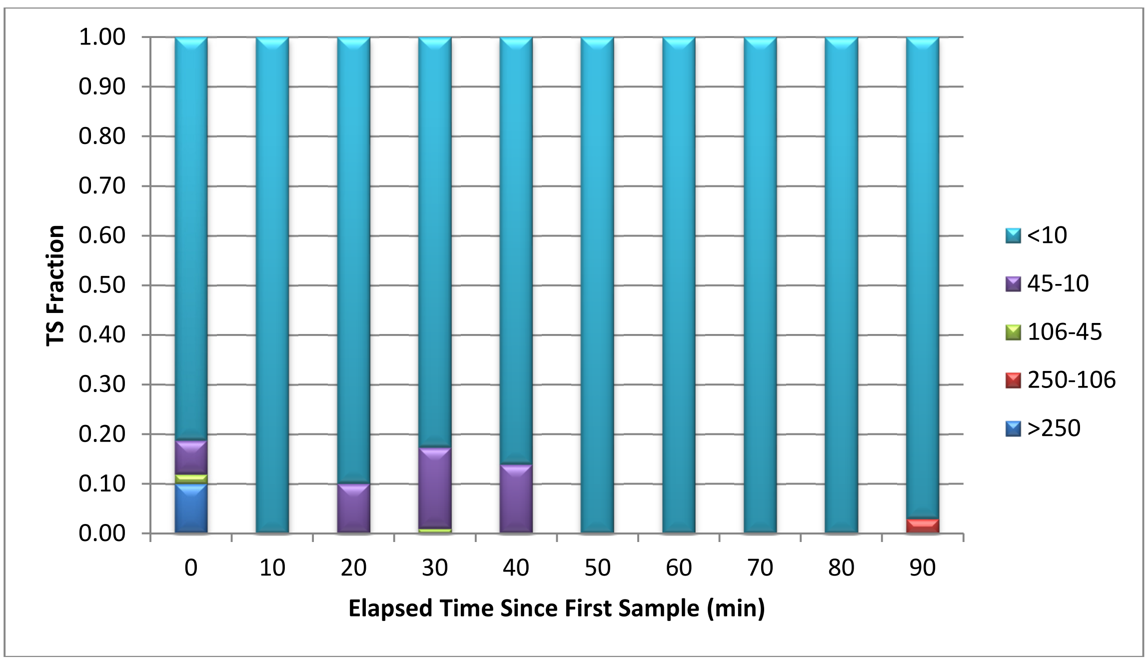


Table 2-13
Normalized TP Fractions by Particle Size in Pipe Flow from Storm III

		Size Range of Solid Particulates						
Time	**Elapsed Time (min)**	**>250**	**250-106**	**106-45**	**45-25**	**25-10**	**10-0.45**	**DP**
11:51	0	0.00	0.00	0.00	0.77	0.12	0.04	0.08
12:05	15	0.00	0.00	0.00	0.73	0.18	0.03	0.06
12:20	30	0.03	0.09	0.00	0.59	0.13	0.09	0.06
12:35	45	0.06	0.00	0.00	0.68	0.15	0.06	0.06
12:50	60	0.15	0.12	0.00	0.42	0.15	0.08	0.08
13:05	75	0.14	0.37	0.02	0.28	0.07	0.07	0.05
13:20	90	0.00	0.18	0.00	0.61	0.08	0.08	0.05
13:35	105	0.05	0.05	0.00	0.68	0.03	0.13	0.05
13:50	120	0.05	0.00	0.03	0.78	0.05	0.05	0.05
14:05	135	0.00	0.00	0.05	0.78	0.05	0.05	0.05
14:20	150	0.00	0.00	0.05	0.81	0.00	0.08	0.05
14:35	165	0.21	0.00	0.08	0.58	0.02	0.06	0.04

Figure 2-22
Normalized TP Fractions by Particle Size in Pipe Flow from Storm III

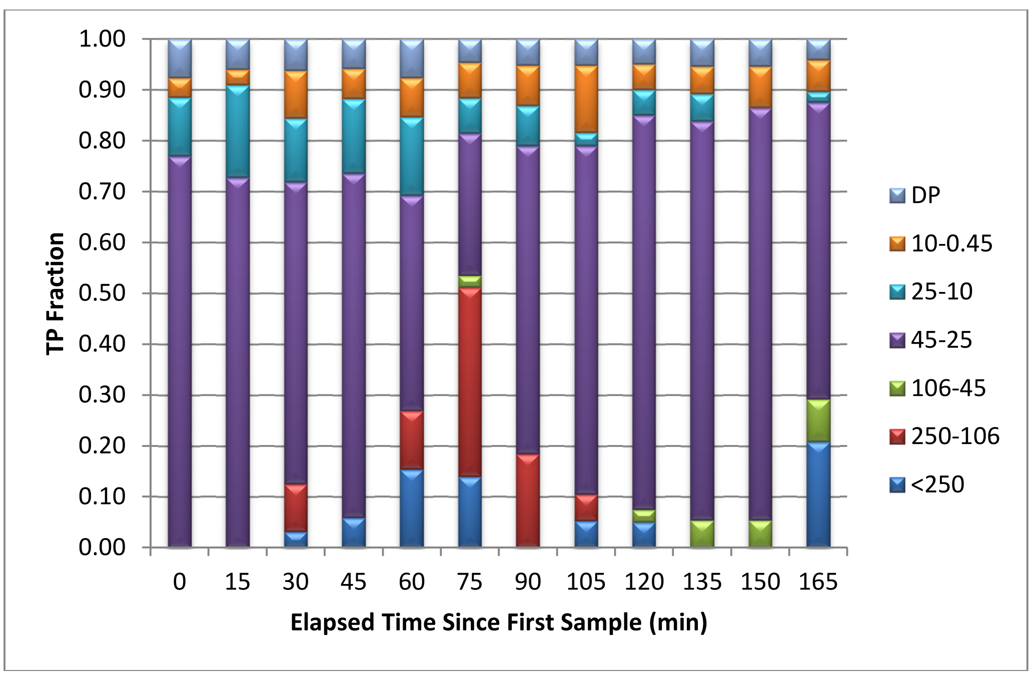
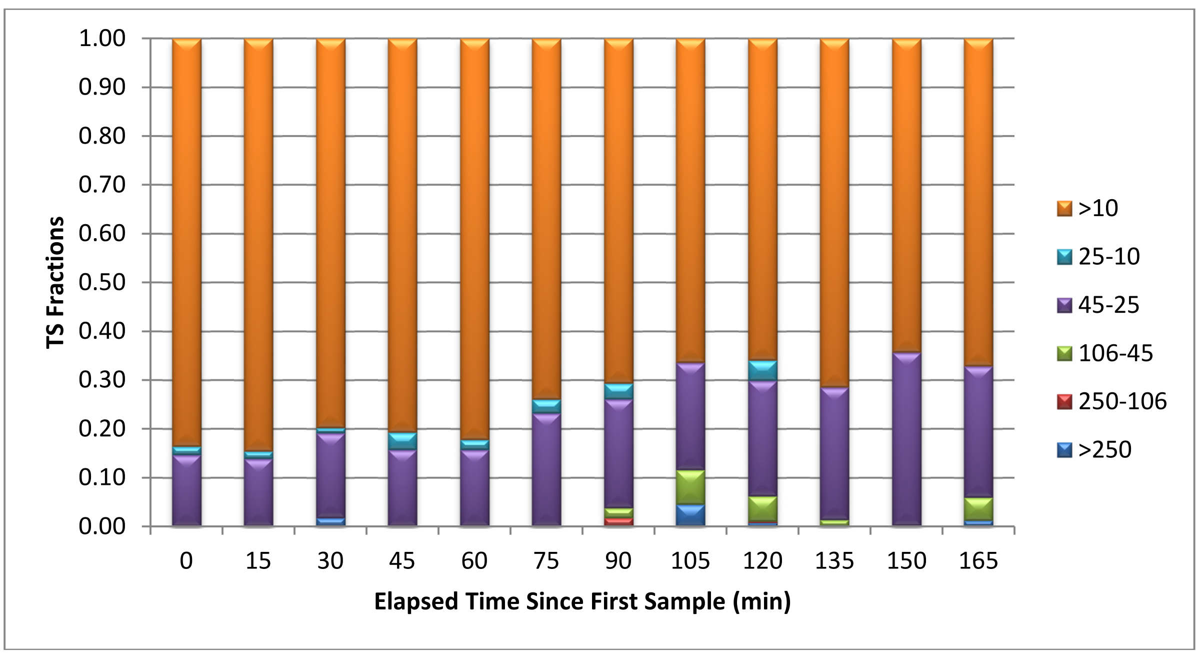


Table 2-14
Normalized TS Fractions by Particle Size in Pipe Flow from Storm III

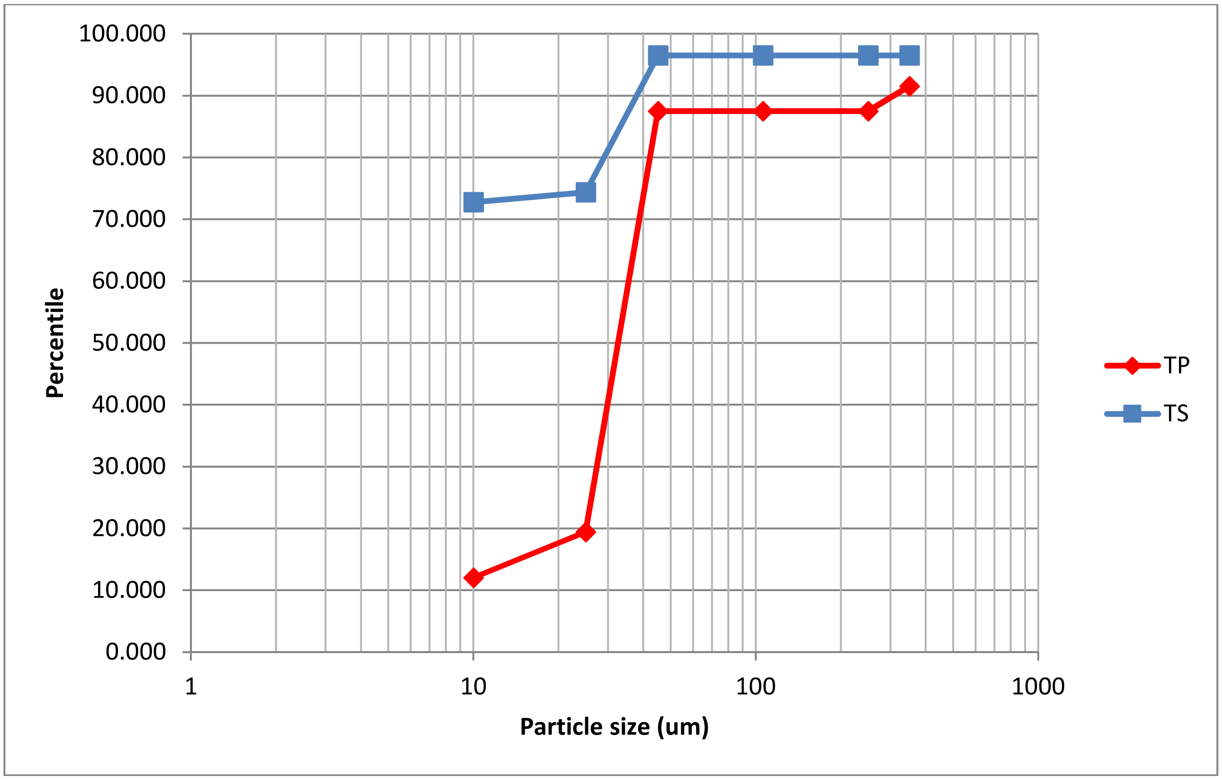
		Size Range of Solid Particulates					
Time	Elapsed Time (min)	>250	250-106	106-45	45-25	25-10	<10
11:51	0	0.00	0.00	0.00	0.15	0.02	0.84
12:05	15	0.00	0.00	0.00	0.14	0.02	0.85
12:20	30	0.02	0.00	0.00	0.17	0.01	0.80
12:35	45	0.00	0.00	0.00	0.16	0.04	0.81
12:50	60	0.00	0.00	0.00	0.16	0.02	0.82
13:05	75	0.00	0.00	0.00	0.23	0.03	0.74
13:20	90	0.00	0.02	0.02	0.22	0.03	0.71
13:35	105	0.05	0.00	0.07	0.22	0.00	0.66
13:50	120	0.01	0.00	0.05	0.24	0.04	0.66
14:05	135	0.00	0.00	0.01	0.27	0.00	0.71
14:20	150	0.00	0.00	0.00	0.35	0.00	0.64
14:35	165	0.01	0.00	0.05	0.27	0.00	0.67

Figure 2-23
Normalized TS Fractions by Particle Size in Pipe Flow from Storm III



The following figure presents the cumulative TS and TP distributions for the stormwater pipe flow collected on storm III. The cumulative percentiles for each particle size range were obtained using median values of the TS and TP fractions presented Table 2-13 and 2-14. A similar figure was not created for storm I because all the captured phosphorus was dissolved as indicated in Table 2-11.

Figure 2-24
Pipe Flow TP and TS Distributions with Respect to Particle Size in Storm III



2.2.3 Statistical Analysis

In order to identify significant differences in TP and TS distributions across different particle size ranges; Mann-Whitney statistical tests of significance at a 95% confidence level between pairs of data sets were performed. In tables 2-15 through 2-17, a *p* value smaller than 0.05 indicates statistically significant differences between pairs of groups.

In storm I, no statistically significant differences were identified among the TP distributions between groups. All the phosphorus seemed to be dissolved. Similarly, no statistically significant differences in TS distribution were identified among pairs of groups. Almost all of the particulates were 10 microns or smaller and likely to be smaller than 0.45 microns (threshold size for dissolved solids); which would be consistent with the TP results.

Table 2-15
Pipe Flow Matrix of Mann-Whittney *p* Values for pairs of TS Distributions in Storm I

	Unfiltered	**<250**	**<106**	**<45**	**<10**
Unfiltered	----	0.6241	0.3077	0.3628	0.8808
<250			0.3271	0.4715	0.7642
<106				0.8181	0.4473
<45					0.5687
<10					----

In storm III, the statistical tests successfully identified five groups with significant differences in TP distributions with respect to the rest: 45 microns or larger, 45 to 25 microns, 25 to 10-microns, 10 to 0.45 microns, and smaller than 0.45 microns (dissolved).

The statistical tests only identified two groups with significantly different TS distribution with respect to the rest. The first group included particles larger than 45 microns, while the second one included particles smaller than 45 microns. Unlike with TP, no significant differences in TS were found between the 25- and 10-micron subsamples. This difference between TP and TS distribution may indicate different phosphorus retention capacities of particles 45-to-25 and 25-micron or smaller in size. Results from the statistical tests are presented in Table 2-17.

Table 2-16
Pipe Flow Matrix of Mann-Whittney p Values for pairs of TP Distributions in Storm III

	Unfiltered	**<250**	**<106**	**<45**	**<25**	**<10**	DP
Unfiltered	----	0.0784	**0.0375**	0.0643	**<0.0001**	**<0.0001**	**<0.0001**
<250			0.3271	0.8181	**<0.0001**	**<0.0001**	**<0.0001**
<106				0.4902	**<0.0001**	**<0.0001**	**<0.0001**
<45					**<0.0001**	**<0.0001**	**<0.0001**
<25						**0.0002**	**<0.0001**
<10							**<0.0001**
DP							----

Table 2-17
Pipe Flow Matrix of Mann-Whittney p Values
for pairs of TS Distributions in Storm III

	Unfiltered	<250	<106	<45	<25	<10
Unfiltered	----	1	0.9522	0.7039	**0.0285**	**0.0121**
<250			0.9283	0.749	**0.0264**	**0.0121**
<106				0.749	**0.0264**	**0.0131**
<45					**0.0264**	**0.0193**
<25						0.9045
<10						----

2.2.4 Discussion and Conclusions

1. Only dissolved phosphorus or phosphorus attached to very small particles (D <10μm) is transported at pipe flow velocities smaller than 0.7-0.75 fps as seen in the TS and TP fraction analysis and the statistical tests of significance. DP represents 100% of the phosphorus load when flow velocity is below this threshold value. This is easy to observe by comparing Figure 2-16 to Figure 2-21. A "delayed wave" of particulates between 10 and 45 microns follows a velocity peak of 0.75 feet per second.

2. Most of the TP (~70% in average) is bound to particles between 25 and 45 microns when pipe flow velocity exceeds 0.75 fps. Under these conditions, around 10% of TP is bound to particles between 25 and 10 microns, and approximately 6% is dissolved. Therefore, particle-bound phosphorus attached to particles smaller than 45 microns and DP represent approximately 85% of the total phosphorus when velocities of 0.75 fps are reached and TP supply is not limiting.

3. TP solid size distribution at flow velocities larger than 0.75fps does not follow the TS solid size distribution. At these flow velocities, particles 10 microns or smaller account for 75% of the TS in average, while particles between 10 to 25 and 25 to 45 microns account for an average 2% and a 22% of the TS respectively. Therefore, this indicates that particles between 25 and 45 microns have a much higher phosphorus-binding capacity than smaller or larger solids. Phosphorus loading curves for different particle size ranges were developed and presented in Appendix A.

Section 3
Description of Available Alternatives

3.1 INFILTRATION PRACTICES

Infiltration practices usually guarantee an on-site treatment of the stormwater by using the soil as a filter able to capture many of the pollutants carried by stormwater. Different technologies exist to enhance infiltration such as pervious pavements, dry wells, swales etc.

However, infiltration is an effective solution only when the soil and water table conditions are adequate. The project area was mostly salt marshes overlaying blue clay in the early 1900s and was progressively filled with urban land fill like many other areas in Cambridge such as Cambridgeport or MIT. This was recently confirmed during a common manhole separation at the intersection of Putnam Avenue and Kinnaird Street which is within the limits of the project area. The blue clay layer expanded between 4 and 25 feet below the ground; while the four-foot thick top layer feet was composed of mostly coarse aggregate. Dewatering of the excavation was not required due to the clay, shown in Figure 3-1, acting as a natural water dam.

MWH's long experience as a consultant for the City of Cambridge has demonstrated that these soils are not adequate for infiltration. Phase I & II of the Infiltration/Inflow and Cambridgeport projects concluded that soils in the area were mostly of hydrologic types C and D; which precludes any significant, large scale infiltration TP control options.   For this reason, infiltration practices were not selected as a possible TP control alternative as they were deemed not feasible.

Figure 3-1
Thick Layer of Blue Clay at Putnam Avenue and Kinnaird Street

3.2 CONVENTIONAL BEST MANAGEMENT PRACTICES

Three conventional BMPs were evaluated as potential solutions to the meet the TP standard. The first one consists of a periodic street sweeping program. The City of Cambridge is currently implementing a street sweeping program in the Western Ave area which involves a monthly sweeping frequency using a combination of mechanical and vacuum sweepers. Street sweeping may help reduce pollutant loadings because a significant part of TP is attached to small solid particles as seen in previous sections.

The second evaluated BMP consisted of rain gardens on the sidewalks of a portion of Western Ave. The maximum realistic extent for rain gardens in the catchment area is around 0.25 acres. The whole area is approximately 92 acres, therefore the rain gardens would cover 0.27% of the drainage area. Rain gardens provide phosphorus control by infiltrating and treating captured water. The treated water is then re-routed to the storm system via an underdrain connected to the drain pipes.

The third BMP consisted of installing catch basins with a deep sump to maximize the capture of solid particles that could contain attached phosphorus by allowing more settling time. Installation of hoods in the deep sump catch basins would help prevent floatables and oils from entering the storm system.

TP removal efficiencies and cost-benefit analyses for each BMP are presented in Section 4 of this report.

3.3 FLOW DEFLECTION ALTERNATIVES

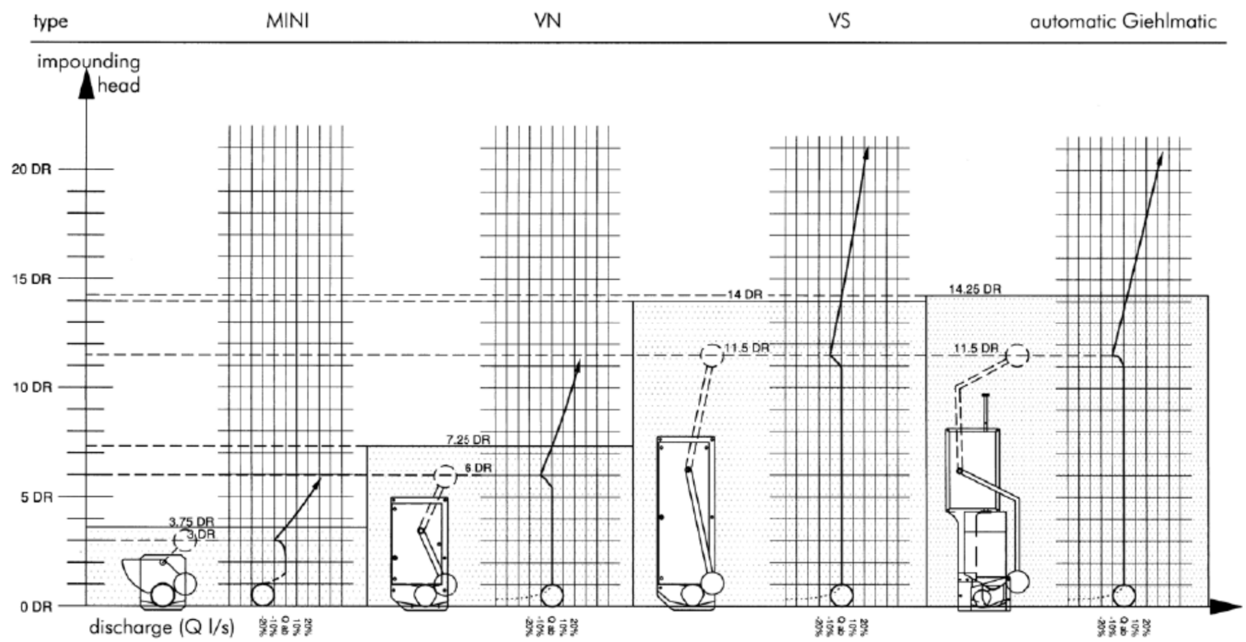
Three different flow deflection alternatives were selected as potential alternatives to reduce TP discharged from the area. The first alternative consists of strategically transferring a portion of the flow to the existing combined sewer which discharges into the MWRA's NCRS. The second alternative would consist of deflecting a portion of the flow to a proprietary filtering system (Jellyfish®). The manufacturer claims the system is able to capture a significant fraction of 25-micron or smaller solid particulates. The third alternative would consist of deflecting a portion of the flow to another proprietary system (Sorbtive™ Filter). The manufacturer claims it is able to treat a portion of both dissolved and particle-bound phosphorus. The treated flow with the second and third deflection alternatives would be conveyed back to the storm system and discharged into the Charles River after treatment.

All three alternatives require the construction of a deflection system regardless of the subsequent stormwater treatment. In order to design this deflection system and be able to estimate the annual deflected volume and associated phosphorus under each scenario, a prototype was developed for the Bishop Allen Drive catchment in Cambridge, MA. Based on the results from the prototype, a final deflection model was created for the Western Ave catchment. Both the prototype and the final model are described in detail in the following section.

3.4 DESCRIPTION OF THE DEFLECTION SYSTEM

The deflection system would consist of a spillway followed by a receiving chamber with a constant flow regulating device or Hydroslide® installed in the chamber outlet pipe. The Hydroslide® is a valve designed to maintain a constant discharge for a hydraulics head up to twelve (12) times the nominal valve inlet diameter (actual value depends on the model). The Hydroslide® progressively increases the discharging capacity by sliding a pivoting gate in front of the inlet valve pipe when the hydraulic head in the valve inlet is smaller than one (1) diameter. This gate is activated mechanically by a float connected to an arm. When the upstream hydraulic head reaches a value of approximately one (1) valve inlet diameter, the discharge is then kept constant until a higher head threshold value is reached, which depends on the Hydroslide® model. Depending on the Hydroslide® model, when this threshold is surpassed, discharge flow either increases following the orifice flow equation (model shown in Figure 3-1) or shuts off completely.

Figure 3-2
Head-Discharge Curves for Different Constant Flow Hydroslide® Sizes



Therefore, deflection of a portion of the flow would start when the water depth in the drain pipe exceeds the spill elevation (low set point) and progressively increases until a set maximum water depth is reached in the receiving chamber (i.e. the hydraulic head in the valve inlet is equal to the inlet pipe diameter); which determines the maximum possible deflection flow rate. If the water level in the chamber is high enough as to reach the hydraulic head high threshold, the deflected flow would then increase (orifice-mode model) or stop (shutoff model).

3.5 PROTOTYPE MODEL DEVELOPMENT

In order to evaluate the performance of the flow controller under different settings, an interactive tool was developed using Microsoft Excel. This "TP Mass Load & Deflection Calculation Tool" was able to compute the deflected stormwater volume and associated TP mass on a yearly basis with different system settings and configurations. TP mass was calculated using observed or estimated first-flush, non-first-flush, and dissolved TP concentrations; duration of first flush conditions; controller low and high set point values; and minimum flow required for particle movement, which is determined by the critical shear stress for particles of a specific size.

In the first version of the tool, it was assumed that the first flush started when a "trigger velocity" was reached. Both "trigger velocity" and duration of first flush conditions are set at will by the user. During this period of time, first-flush TP concentrations were assumed. Non-first flush TP concentrations were assumed once the first flush ended and dissolved TP concentrations were assumed when the flow velocity fell below the "trigger velocity".

The second version of the tool assumed first flush occurred when fluid shear stress fell between the minimum necessary to move 35-micron particles and a higher shear stress set point. The high set point was set manually by the user. All particles between 25 and 45 microns were assumed to be flushed away when a specific high shear stress value is reached. A detailed description of shear stress calculations is presented in Section 3.5.2.

3.5.1 Hydrographs and Hydraulic Properties

To simplify the development of the initial prototype, the rational method (Equation 3-1) was used to construct the hydrograph for each storm type.

$$Q_p = i_{max} \times C \times A \qquad \textbf{Equation 3-1}$$

Where Q_p is the peak flow in cubic feet per second, i_{max} is the average maximum rainfall intensity (in inches/h) during a period of time equal or greater than the time of concentration, C is the runoff coefficient, and A is the basin area in acres.

Since the maximum average rainfall intensity (i_{max}) and the maximum flow (Q_p) for the 1-month MWRA storm were known (i_{max} = 0.33 in/h and Q_p = 2.43 cfs respectively), the $C \times A$ can be calculated and is equal to 7.36. Assuming this parameter does not change with different storm types and varying hydraulic conditions, the peak flow values for the storms in Table 3-2 can be calculated.

Even though the rational method represents a vast simplification of the actual system hydraulics, this was initially used for the evaluation of different flow deflection alternatives for the prototype in Bishop Allen Drive. Hydraulic properties and assumed TP concentrations in this catchment are presented in Table 3-1. The actual and rational hydrographs for storm III in this catchment area are presented in Figure 3-2. The estimated rational peak flow was in strong agreement with the observed peak flow (1.62 versus 1.70 cfs, respectively). The rainfall intensity during the hour prior to the observed peak flow (i_{max}) averaged 0.22 in/h.

Figure 3-3
Actual vs. Rational Method Hydrograph for Storm III

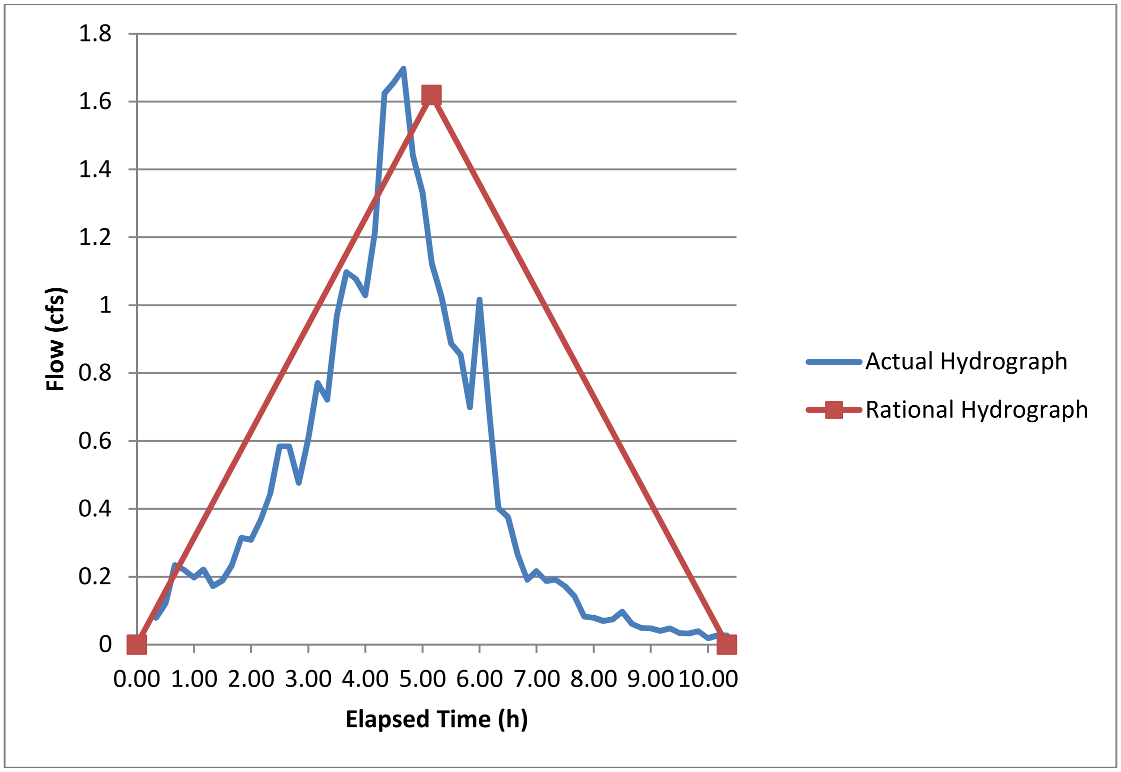


Table 3-1
Prototype Deflection Model Hydraulic and System Properties

Characteristic	Values
First Flush TP Concentration (mg/L)	0.45
Non-First Flush TP Concentration (mg/L)	0.22
DP Concentration (mg/L)	0.06
Slope (ft/ft)	0.0025
Roughness	0.017
Pipe Diameter (ft)	2.00
CA Value	7.36

3.5.2 Shear Stress with Respect to Particle Size

Solid particles sitting at the invert of a storm drain will be suspended and conveyed when the boundary shear stress exerted by the fluid is enough to overcome the critical stress necessary to scour a particle of a specific size from the pipe bed. Shear stress is a function of the fluid physical properties and the system hydraulic characteristics.

The average boundary shear stress can be calculated with Equation 3-2:

$$\tau = \left\{\frac{n^2}{R^{1/3}}\right\} \times \gamma \times V^2 \qquad \textbf{Equation 3-2}$$

Where:

τ = shear stress (N/m^2)
γ = fluid specific weight (N/m^3)
R = hydraulic radius (m)
V = flow velocity (m/sec)

For channels with less than 10° of slope, the mean boundary shear can be approximated with the following equation:

$$\tau = \rho \times g \times R_h \times Slope \qquad \textbf{Equation 3-3}$$

Where:

τ = shear stress (N/m^2)
ρ = fluid density (kg/m^3)
g = gravitational acceleration (m/s^2)
R_h = hydraulic radius (m)
Slope = slope of the drain (dimensionless)

As stated previously in this report, most of the particle-bound phosphorus (which represents most of the TP) seems to be concentrated in particles between 25 and 45 microns. Our observations seem to indicate that particles of this size start moving when a velocity of approximately 0.75 fps is reached. This value agrees quite well with the velocity value calculated with the following equation, commonly used to estimate necessary shear stress for erosion of particles of a specific size.

$$\tau_{cr,erosion} = \gamma_{erosion} \times g \times (s-1) \times \rho \times d_{50}/1000 \qquad \textbf{Equation 3-4}$$

Where:

$\gamma_{erosion}$ = Erosion parameter (usually around 1; dimensionless)
g = gravitational acceleration (m/s^2)
s = specific sediment density (dimensionless)
ρ = fluid density (Kg/m^3)
d_{50} = sediment particle size (mm)

According to Equation 3-4, and assuming $\gamma_{erosion}$ equal to 1 and s = 2.0 (median specific gravity in stormwater systems according to literature), a 35-micron particle would start moving when a shear stress equal or greater than 0.343 N/m^2 is reached. By plugging this number into Equation 3-3 and solving, an R_h equal to 0.014 meters is obtained using the slope in the Bishop Allen Drive manhole. This hydraulic radius corresponds to a flow velocity equal to 0.56 fps in the Bishop Allen system. Therefore, most of the phosphorus (which is bound to particles between 25 and 45 microns) should start moving when a flow velocity of 0.56 fps is reached. It is important to keep in mind that this a conservative value because Equation 3-4 assumes non-cohesive behavior of solid particles. Particulates between 25 and 45 microns are within the silt size range (2µm -60µm) which, according to literature, may still have significant cohesive properties. This cohesive behavior would result in higher velocities required to move 25 to 45-micron particles which would explain the difference between the observed and calculated minimum velocities for particle-bound TP movement.

3.5.3 Phosphorus First Flush and Phosphorus Supply

The existence of a first flush of pollutants in the initial stages of a storm has been extensively documented in many research efforts. A phosphorus first flush will be observed when enough energy exists to release and convey solids since phosphorus is mostly bound to particles as explained in previous sections. This energy is initially provided by rain drops (impact energy) and subsequently by the surface or pipe gradient which generates sheet or pipe flow (shear stress) which suspends solid particles.

One of the main challenges when analyzing total phosphorus loading for a particular catchment is determining the duration of the first flush or non-limiting supply phase. It seems clear that after a period of time with runoff flowing in the streets, phosphorus supply should decrease because most of it has been flushed away. First flush duration is hard to estimate accurately as it is a function of many intertwined factors such as rainfall intensity and distribution, catchment characteristics, buildup time between consecutive storms, presence of BMPs, etc.

The duration of the phosphorus first flush could not be estimated in the pipe flow sampling events. Pipe flow phosphorus results from samples collected during storm I indicated that transport of particles smaller than 45 microns only started when flow velocities exceeded 0.75 fps approximately. Pipe flow samples collected during storm III were collected after a set "trigger velocity" of 0.75 fps was reached. When the first sample was collected, 45-micron particles were already moving and carrying a large percentage of the TP. During the next 2 hours and 45 minutes more samples were collected and the phosphorus supply didn't seem to wind down. The overall TP concentration in storm III averaged 0.36 mg/L (unfiltered sample). This value is a clear indication that the supply was still abundant and the first flush period had not ended since reported TP Event Mean Concentrations (EMC) for high density residential land uses is around 0.31 mg/L according to Pitt (2004).

3.5.4 Prototype Results

Year-long simulations were performed for an average year (46.8 inches of rain), for a year with more rainfall than average (53.2 inches of rain [year 2008]), and for a dry year (43 inches). Typical storms within a one-year period were split into five types (Table 3-2).

Table 3-2
Properties of Different Storm Types in a Typical Year

Storm Type	Average intensity (in/h)	Average Rainfall Depth (in)	Average Duration (h)	Peak Flow (cfs)-Rational Method	Number of Storms in Wet Year	Number of Storms in a Normal Year	Number of Storms in a Dry Year
1	0.025	0.075	3	0.39	47	47	46
2	0.039	0.350	9	1.40	24	24	21
3	0.054	0.685	12.75	2.13	15	13	13
4	0.070	1.574	22.40	4.78	10	9	9
5	0.080	3.052	37.90	3.83	5	4	3

Calculation of Deflected TP Mass Based on First Flush Duration

Results from the first version of the Excel tool are presented in Table 3-3 through Table 3-5. A first-flush duration of 4 hours was assumed. The minimum velocity to move 35-micron particles was set to 0.70 fps since the experiment results showed no particle movement occurred below this point. The following tables present results with a 0.16-feet high spill level.

Table 3-3
Deflected Flow and TP with Different System Settings in a Wet Year Based on First Flush Duration

Scenario #	Spill Level (ft)	Max. Flow Diversion (cfs)	Volume Diverted (MG)	% of Total Volume Deflected	Total TP load (kg/y)	Total TP Diverted (kg/y)	% of TP Diverted
1	0.16	0.75	11.95	34.58	32.89	8.02	24.37
2	0.16	1.00	15.47	44.78	32.89	10.29	31.30
3	0.16	1.50	20.82	60.25	32.89	13.81	41.98
4	0.16	2.00	24.58	71.14	32.89	16.34	49.68

Table 3-4
Deflected Flow and TP with Different System Settings in an Average Year Based on First Flush Duration

Scenario #	Spill Level (ft)	Max. Flow Diversion (cfs)	Volume Diverted (MG)	% of Total Volume Deflected	Total TP load (kg/y)	Total TP Diverted (kg/y)	% of TP Diverted
1	0.16	0.75	10.67	35.08	29.19	7.17	24.55
2	0.16	1.00	13.77	45.28	29.19	9.16	31.39
3	0.16	1.50	18.41	60.52	29.19	12.20	41.79
4	0.16	2.00	21.64	71.15	29.19	14.37	49.21

Table 3-5
Deflected Flow and TP with Different System Settings in a Dry Year Based on First Flush Duration

Scenario #	Spill Level (ft)	Max. Flow Diversion (cfs)	Volume Diverted (MG)	% of Total Volume Deflected	Total TP load (kg/y)	Total TP Diverted (kg/y)	% of TP Diverted
1	0.16	0.75	9.76	34.95	26.89	6.54	24.34
2	0.16	1.00	12.60	45.10	26.89	8.36	31.01
3	0.16	1.50	16.85	60.32	26.89	11.12	41.34
4	0.16	2.00	19.81	70.91	26.89	13.08	48.65

Calculation of Deflected TP Mass Based on Shear Stress

Deflected flow and TP percentages were calculated using the second version of the Excel tool. A shear stress bracket ranging from 0.3 N/m^2 (start of first flush) to 2.0 N/m^2 (end of first flush) was used. Different prototype configurations and respective performances are presented in Table 3-6 through Table 3-8.

Table 3-6
Deflected Flow and TP with Different System Settings in a Wet Year Based on Shear Stress

Scenario #	Spill Level (ft)	Max. Flow Diversion (cfs)	Volume Diverted (MG)	% of Volume Diverted	Total TP load (kg/y)	Total TP Diverted (kg/y)	% of TP Diverted
1	0.16	0.75	11.95	34.58	35.30	6.83	19.36
2	0.16	1.00	15.47	44.78	35.30	8.75	24.79
3	0.16	1.50	20.82	60.25	35.30	11.67	33.05
4	0.16	2.00	24.58	71.14	35.30	13.73	38.90

Table 3-7
Deflected Flow and TP with Different System Settings in an Average Year Based on Shear Stress

Scenario #	Spill Level (ft)	Max. Flow Diversion (cfs)	Volume Diverted (MG)	% of Volume Diverted	Total TP load (kg/y)	Total TP Diverted (kg/y)	% of TP Diverted
1	0.16	0.75	10.67	35.08	31.30	6.14	19.71
2	0.16	1.00	13.77	45.28	31.30	7.83	25.12
3	0.16	1.50	18.41	60.52	31.30	10.36	33.24
4	0.16	2.00	21.64	71.15	31.30	12.14	38.94

Table 3-8
Deflected Flow and TP with Different System Settings in a Dry Year Based on Shear Stress

Scenario #	Spill Level (ft)	Max. Flow Diversion (cfs)	Volume Diverted (MG)	% of Volume Diverted	Total TP load (kg/y)	Total TP Diverted (kg/y)	% of TP Diverted
1	0.16	0.75	9.76	34.95	28.79	5.65	19.72
2	0.16	1.00	12.60	45.10	28.79	7.19	25.12
3	0.16	1.50	16.85	60.32	28.79	9.52	33.24
4	0.16	2.00	19.81	70.91	28.79	11.14	38.91

3.6 FINAL FLOW DEFLECTION MODEL FOR THE WESTERN AVE AREA

In the final model, the rational method was no longer used to build the site's hydrographs. In this case, hydrographs for the different storm types in Table 3-2 were generated with Infoworks. In order to model each storm, synthetic hyetographs with a total rainfall depth equal to the average recorded storm depths in Table 3-2 were generated. A symmetrical or quasi-symmetrical shape was assumed.

The system was modeled in two different locations to make sure all the flow leaving the existing (Flagg Street) and projected (Western Ave) separated subcatchments could be treated. The first location was at the intersection between Western Ave and Jay Street and the second one was near the intersection between Flagg Street and Memorial Drive. The performances of two types of Hydroslides were evaluated. The first model consisted of a slide that guaranteed constant flow at the outlet when the hydraulic head fell between a specific range of values (Figure 3-1). The second model consisted of a Hydroslide that guaranteed a constant outlet flow until a threshold head value was reached. When this occurred, the slide would completely shut until the hydraulic head fell below the threshold again. An example of the deflection behavior of each Hydroslide model for the same storm is presented in Figure 3-4 and Figure 3-5. Results from the final model are presented in the following section where TP removal efficiencies and costs for the different alternatives are evaluated.

The shear stress approach was selected over the first flush duration one and used for the final model development because it was deemed that the duration of the first flush phase for this approach could be estimated more accurately. The end of first flush shear stress is not only a function of the particle size but also the hydraulic properties of the study system and the rainfall distributions.

With the shear stress approach, TP concentrations for different phases of the storm –dissolved, first flush and non-first-flush- were assigned based on literature values and lab results (0.06, 0.45, and 0.22 mg/L respectively). The start and end times of the different storm phases were set based on shear stress values. The first flush started when the shear stress at the point of flow deflection was large enough to move 25-micron particles (i.e. 0.34 N/m^2) and ended when a shear stress value of 1N/m^2 was reached. At that point, it was considered that all upstream particles between 25 and 45 microns, which contain most of the phosphorus, had been flushed away. The end of first flush value was selected after making sure that velocities upstream of the deflection systems guaranteed the minimum shear stress was reached. Velocities in the Western Ave catchment reached values of 1fps for small storms and up to 7fps for larger storms. Approximately 0.9 fps were needed in our system to achieve a shear stress of 0.34N/m^2.

Figure 3-4
Typical Flow and Shear Stress Curves with a Constant Flow Hydroslide®

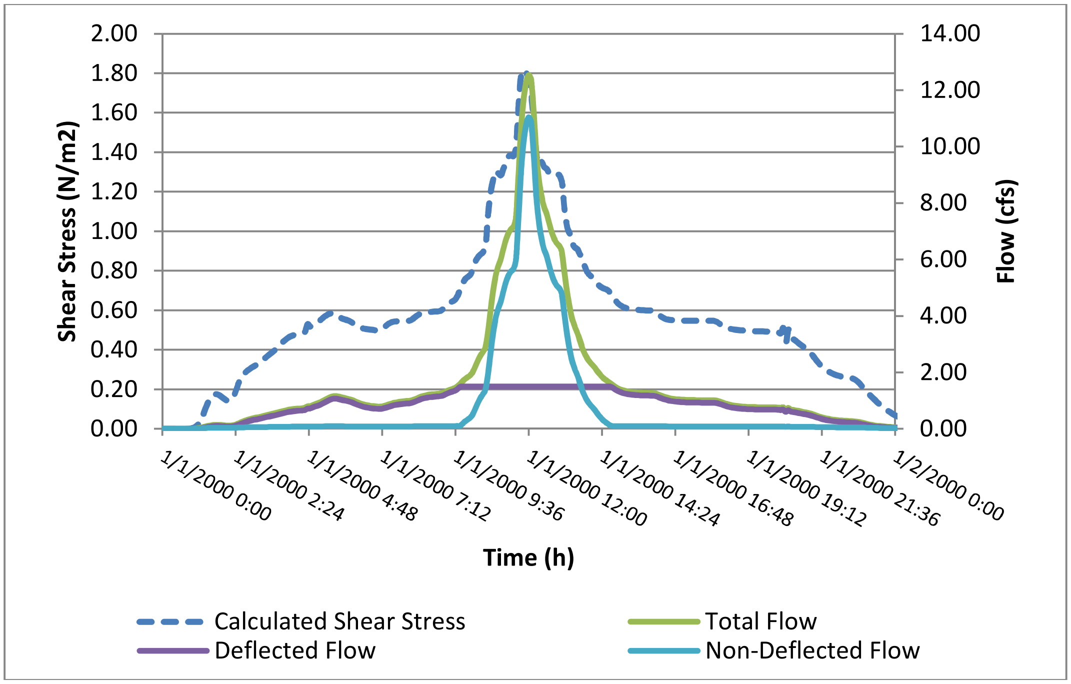
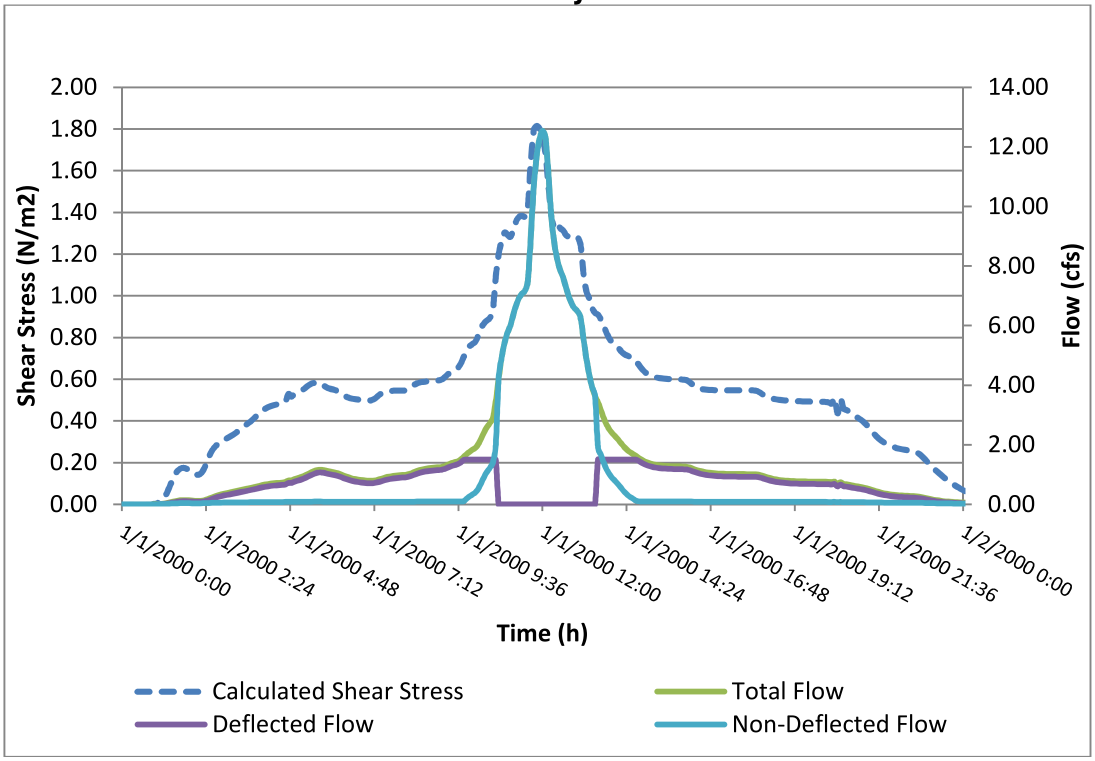


Figure 3-5
Typical Flow and Shear Stress Curves with a Constant Flow Hydroslide® with Shutoff System



Section 4
Cost-Benefit Analysis of Alternatives

4.1 SELECTED ALTERNATIVES

Different combinations of alternatives that could potentially be implemented in the catchment area in order to meet the required TP reduction were analyzed. Calculations of present worth of costs for the next twenty (20) years as well as the TP removal efficiencies were computed.

The TP removal efficiencies and cost calculations were performed for the following alternatives:

1 Conventional BMPs
2 Conventional BMPs + deflection to MWRA
3 Conventional BMPs + deflection to a Jellyfish System
4 Conventional BMPs + deflection to a Sorbtive System

All cost calculations for alternatives 2,3 and 4 were based on selecting the system configuration that guaranteed the minimum possible deflected volume necessary to meet the 65.2% TP reduction after implementation of conventional BMPs. If the required 65.2 % TP removal could not be achieved with BMP + deflection then, the cost calculations were based on maximum TP removal possible.

4.2 ASSUMPTIONS AND COST CALCULATION METHODOLOGY

In order to estimate the TP reduction for the different proposed alternatives, the following efficiencies were used based on available literature and reported equipment performances.

Table 4-1
TP Removal Efficiencies for Proposed BMPs

BMP	TP Removal Efficiency	Comment
Rain gardens	100% (self-contained) 73% (under-drained)	Percent of TP removed for captured flows only TP removal rates from Schilling, J.G. (2005)
BMP catch basins	2% (assumed)	Percent of TP removed for captured flows only
Vaccum-street sweeping	8% (Weekly) 6% (Bi-weekly) 4% (Monthly)	1 pass assumed during each street cleaning event TP removal rates from Center for Watershed Protection (2008)
Mechanical Street Sweeping	5% (Weekly) 4% (Bi-weekly) 3% (Monthly)	1 pass assumed during each street cleaning event TP removal rates from Center for Watershed Protection (2008)
Jellyfish™	40% (pessimistic) 60% (optimistic)	Assuming 0% removal efficiency of particles smaller than 25 microns and 50 and 75% removal efficiency of particles between 25 and 45 microns (80% of TP). Max. flow of 2 cfs.
Sorbtive Filter™	50% (pessimistic) 70% (optimistic)	Assumed 15% less removal efficiency than reported by manufacturer. Maximum flow of 1.65 cfs

Other assumptions used are listed as follows:

- Costs are based on an average year in terms of rainfall depth
- A maximum of 0.25 acres of rain gardens is feasible in the Western Ave catchment (0.27% of the total area, which has 92 acres)
- Rain gardens will capture and treat 90% of the flow they receive
- Interest rate is constant and equal to 5%
- Inflation rate is constant and equal to 3%
- Costs were estimated based on MWH experience, information supplied by the City of Cambridge, or already executed similar projects in other municipalities

The present worth of costs in the next twenty (20) years for each alternative was calculated using the following equation:

$$PV\ (20\ years) = Installation\ cost + \sum_{i=0}^{20} \frac{Annual\ O\&M\ Cost}{(1+Net\ Interest)^i}$$ **Equation 4-1**

Where:

PV = present value of costs
Installation cost = cost of installation in 2010 dollars
Annual O&M Cost = annual O&M costs in 2010 dollars
i = number of years
Net interest $= \frac{Interest\ rate\ (\%) - Inflation\ (\%)}{100}$

4.3 ALTERNATIVE 1: CONVENTIONAL BMP'S

The estimated TP load removal with the implementation of conventional BMPs (rain gardens, vacuum or mechanical street sweeping, deep sump catch basins)

4.3.1 Estimated TP Removal

Rain Gardens:

$$\begin{aligned}\text{TP Reduction by Rain Gardens} &= \%\,\text{Total Area} \times \%\,\text{Treated Flow} \times \text{Removal Efficiency}\\ &= (0.0027 \times 0.90 \times 0.73) \times 100 = 0.18\%\end{aligned}$$

Where:
Area: Fraction of the total drainage area covered by rain gardens
Treated Flow: fraction of the total annual flow entering the rain garden being treated
Removal Efficiency = Rain Garden TP Removal Efficiency

Note: the TP removal calculations for rain gardens are based on the conservative assumption that only runoff from the rain-garden footprint area is being captured. Removal efficiencies may vary between 1 and 3% depending on the contributing area the proposed configuration is capable of treating.

Street Sweeping:

The TP removal efficiencies (in %) for low and high efficiency sweepers and frequencies are presented below.

Table 4-2
Percent TP Removal Efficiencies for Different Street Sweepers and Sweeping Frequencies

	Weekly	Bi-weekly	Monthly
Mechanical Sweeper	5.0	4.0	3.0
Dry-Vacuum Sweeper	8.0	6.0	4.0

Data from the Center for Watershed Protection (2008)

Deep Sump catch Basins: the assumed TP removal is 2% for this BMP

Final Maximum TP Removal:

$$\text{Max. TP Removal } (\%) = 0.18\% + 8\% + 2\% = 10.18\%$$

Projected TP Removal with monthly sweeping frequencies (%) = 0.18% + 4% + 2% = 6.18%

4.3.2 System Installation and Operating Costs

Rain Gardens:

$$\textbf{Installataion Cost} = 10{,}890 \text{ sq. ft} \times \$122/\text{sq. ft} = \$1{,}328{,}580$$

$$\text{Annual } O\&M\ Cost\ (5\%\ of\ construction\ cost) = 0.05 \times \$1{,}328{,}580 = \$66{,}429$$

$$\textbf{Present Cost (20 years)} = \$1{,}328{,}580 + \sum_{i=0}^{20} \frac{\$66{,}429}{(1+0.02)^i} = \$2{,}481{,}218$$

Data source: Weiss et al. (2005)

Street Sweeping:

Annual O&M Cost (Mechanical Sweeper, monthly frequency, 16 curb-miles, $7,000 of disposal per year) $= \$83/\text{curb mile} \times 16 \text{ curb miles} \times 12 \text{ times} + \$7{,}000 \text{ (disposal)} = \$22{,}936$

Present Cost (20 years) = $\sum_{i=1}^{20} \frac{\$22,936}{(1+0.02)^i}$ = \$397,972

Data Source: O&M costs from City of Cambridge, MA

Deep Sump Catch Basins:

It was assumed that 75% of the existing catch basins need to be modified. There are approximately 300 catch basins in the Western Ave area.

$\text{Installation Cost} = (300 \times 0.75) \text{ modified catch basins} \times \$8{,}500/\text{unit} = \$1{,}912{,}500$

Annual O&M Cost (2 times a year, 1 hour per CB, 2 people, clamshell truck) =
$300 \text{ catch basins} \times 2 \text{ times} \times 1\text{hour} \times (2 \text{ people} \times \$80/\text{h} + 1\text{truck} \times \$100/\text{h}) + \$300 \text{ of disposal}/\text{truck} \times 300 \text{ catch basins} \times 2 \text{ times} \times 1 \text{ truck}/30 \text{ catch basins} = \$162{,}000$

Present Cost (20 years) = \$1,275,000 + $\sum_{i=1}^{20} \frac{\$162{,}000}{(1+0.02)^i} = \$4{,}723{,}432$

Data source: MWH experience

Present Cost for all BMP Installation and O&M for the next 20 years:

Total Present Cost (20 years) = \$2,481,218 +\$397,972+\$4,723,432 = \$7,602,622

4.4 ALTERNATIVE 2: CONVENTIONAL BMPS AND DEFLECTION TO THE MWRA SYSTEM

4.4.1 Estimated TP Removal

$\text{TP reduction with BMPs (\%)} = 6.18\%$ (see previous section).

TP removal efficiencies with different Hydroslide® models and system settings are presented in the following tables:

Table 4-3
Flow Deflection and TP Removal Performances for Different System Settings Using a Constant Flow Hydroslide

Location	Spill Elevation (feet)	Max Q (cfs)	Annual Deflected Volume (MG)	% TP Removed
Western Ave @ Jay Street	0.05	1.30	17.57	58.93
Western Ave @ Jay Street	0.10	1.45	17.53	59.52
Western Ave @ Jay Street	0.15	1.60	17.11	58.93
Western Ave @ Jay Street	0.20	1.95	17.11	59.33
Western Ave @ Jay Street	0.25	2.50	17.15	59.09
Western Ave @ Jay Street	0.30	3.75	17.59	58.96
Flagg Street near Memorial Dr.	0.10	1.30	4.89	59.21
Flagg Street near Memorial Dr.	0.20	1.40	4.22	59.8
Flagg Street near Memorial Dr.	0.30	1.55	3.37	59.37
Flagg Street near Memorial Dr.	0.40	2.00	2.65	59.43

***Shaded rows indicate system configurations that meet the 65.2% TP reduction with minimum flow deflection in each area**

Table 4-4
Flow Deflection and TP Removal Performances for Different System Settings Using a Constant Flow Hydroslide with Shutoff System

Location	Spill elevation (ft)	Hydraulic Head (# of diameters)	Hydroslide Max. Deflection Flow (cfs)	Annual Deflected Volume (MG)	% TP Removed
Western Ave @ Jay Street	0.05	1.32	3	17.58	59.06
Western Ave @ Jay Street	0.05	1.8	2	17.00	59.33
Western Ave @ Jay Street	0.10	1.45	3	17.41	59.22
Western Ave @ Jay Street	0.10	2.10	2	16.67	59.01
Western Ave @ Jay Street	0.15	1.65	3	17.21	59.37
Flagg St near Memorial Dr.	0.05	1.30	3	5.16	59.98
Flagg St near Memorial Dr.	0.05	1.75	2	5.14	59.33
Flagg St near Memorial Dr.	0.10	1.30	3	4.29	59.21
Flagg St near Memorial Dr.	0.10	1.80	2	4.88	58.96
Flagg St near Memorial Dr.	0.15	1.35	3	4.59	59.76
Flagg St near Memorial Dr.	0.15	1.90	2	4.55	58.79
Flagg St near Memorial Dr.	0.20	1.40	3	4.22	59.80
Flagg St near Memorial Dr.	0.20	2.20	2	4.19	59.14
Flagg St near Memorial Dr.	0.25	1.45	3	3.79	59.24
Flagg St near Memorial Dr.	0.25	n/a	2	n/a	n/a
Flagg St near Memorial Dr.	0.35	1.70	3	2.97	59.22
Flagg St near Memorial Dr.	0.40	2.65	3	2.65	59.43

***Shaded rows indicate system configurations that meet the 65.2% TP reduction with minimum flow deflection in each area**

Therefore, the total TP reduction will be:

$$\text{Total TP Removal (\%)} = 6.18 + \frac{(31.59 \times 59.01 + 59.43 \times 6.66)}{(31.59 + 6.66)} = 6.18 + 59.1 = 65.3\%$$

Where 31.59 and 6.66 are the millions of gallons of stormwater runoff generated in the Western and Flagg street areas respectively.

4.4.2 System Installation and Maintenance Costs

The cost estimates are best using the optimum system settings for each location (Western Ave and Flagg Street). The selected system settings for each location are shaded in Table 4-3 and Table 4-4.

An MWRA I/I fee of $1,450 per MG was assumed.

Bi-monthly inspections of the structures with periodic flushing was assumed.

Installation cost:

Table 4-5
Estimated Installation Costs of the Hydroslide Deflection System

Item	Number of units	Unit price	Total
6' manhole	4	$16,000	$64,000
Hydroslide	2	$25,000	$50,000
Grit chamber	2	$15,000	$30,000
Backwater flap valves	2	$3,000	$6,000
Total			**$150,000**

O&M Costs:

Annual I/I cost = $(16.67\text{MG} + 2.65\text{MG}) \times \$1{,}450 \text{ per MG} = \$28{,}014$

Annual Maintenance Costs

Cost of each inspection = $3\text{hours} \times (3 \text{ people} \times \$80/\text{h} + 1 \text{ truck} \times \$100/\text{h}) + \$300 \text{ for disposal} = \$1{,}320$

Annual cost = $\$1{,}320 \times 6 \text{ inspections} \times 2 \text{ units} = \$15{,}840$

Total O&M cost = $28,014 + $15,840 = $43,854

Present Cost for the next 20 years:

$$\text{Present Cost for Deflection} = \$150{,}000 + \sum_{i=1}^{20} \frac{\$43{,}854}{(1+0.02)^i} = \$910{,}930$$

$$\text{Present Cost for BMPs} + \text{Deflection to MWRA} = \$7{,}602{,}622 + \$910{,}930 = \$8{,}513{,}552$$

4.5 ALTERNATIVE 3: CONVENTIONAL BMP AND DEFLECTION TO A JELLYFISH™ SYSTEM

4.5.1 Estimated TP Removal

$\text{TP reduction with BMPs } (\%) = 6.18\%$ (see BMP section).

Calculations are based on 10-foot diameter manholes containing a Jellyfish System with 16 standard cartridges (50 gpm each) and 3 draindown cartridges (25gpm each).Therefore, the maximum treatment flow is equal to 875gpm or 1.95cfs.

System performances were calculated using two assumed TP removal efficiencies: 40% (conservative) and 60% (optimistic). Assumptions on Jellyfish TP removal efficiencies are described in Section 4-2. Results for different system settings, TP removal efficiencies, and Hydroslide® models are shown in the following tables.

Table 4-6
Flow Deflection and TP Removal Performances for Different System Settings Using a Constant Flow Hydroslide® Followed by a 2-cfs Jellyfish™ System

Location	Spill elevation (ft)	Hydroslide max. deflection flow (cfs)	Annual deflected volume (MG)	% TP deflected	% TP removed (40% efficiency)	% TP removed (60% efficiency)
Western @ Jay Street	0	2	21.61	72.15	28.80	43.29
Western @ Jay Street	0.05	2	21.06	70.63	28.25	42.378
Western @ Jay Street	0.1	2	20.05	67.83	27.13	40.698
Western @ Jay Street	0.15	2	18.74	64.19	25.68	38.514
Western @ Jay Street	0.2	2	17.28	59.86	23.95	35.916
Western @ Jay Street	0.25	2	15.73	54.95	21.98	32.97
Western @ Jay Street	0.3	2	14.11	49.43	19.77	29.658
Flagg Street	0	2	6.01	80.84	32.33	48.504
Flagg Street	0.05	2	5.85	80.08	32.03	48.048
Flagg Street	0.1	2	5.55	78.64	31.45	47.184
Flagg Street	0.15	2	5.15	76.62	30.65	45.972
Flagg Street	0	1	4.88	48.15	19.26	28.89
Flagg Street	0	1.25	5.25	58.62	23.45	35.172

***Shaded rows indicate system configurations that guarantee the maximum possible TP reduction**

Table 4-7
Flow Deflection and TP Removal Performances for Different System Settings Using a Hydroslide® with Shutoff Followed by a 2-cfs Jellyfish™ System

Location	Spill elevation (ft)	Hydraulic Head (# of diameters) before shutoff	Hydroslide Max. Deflection Flow (cfs)	Annual Deflected Volume (MG)	% TP deflected	% TP removed (40% efficiency)	% TP removed (60% efficiency)
Western Ave @ Jay Street	0	2	2	18.04	62.63	25.05	37.578
Western Ave @ Jay Street	0	2	3	21.45	71.73	28.69	43.038
Western Ave @ Jay Street	0.1	2	2	16.48	58.31	23.32	34.986
Western Ave @ Jay Street	0.1	2	3	19.89	67.41	28.96	40.446
Western Ave @ Jay Street	0.2	2	2	13.71	50.34	20.14	30.204
Western Ave @ Jay Street	0.2	2	3	17.12	59.45	23.78	35.67
Flagg Street	0	2	2	5.44	64.27	25.71	38.562
Flagg Street	0	2	3	6.01	80.84	32.33	48.504
Flagg Street	0.05	2	2	5.28	63.51	25.40	38.106
Flagg Street	0.05	2	3	5.85	80.08	32.03	48.048
Flagg Street	0.1	2	2	4.98	62.06	24.82	37.236
Flagg Street	0.1	2	3	5.55	78.64	31.45	47.184

Therefore, the maximum possible TP removal with the optimistic and the conservative scenarios would be as follows:

1. Optimistic scenario (60% TP removal efficiency)

$$\text{Total TP removal (\%)} = 6.18 + \frac{(43.3 \times 31.59 + 48.5 \times 6.66)}{(31.59 + 6.66)} = 50.4\%$$

2. Conservative scenario (40% TP removal efficiency)

$$\text{Total TP removal (\%)} = 6.18 + \frac{(28.8 \times 31.59 + 32.3 \times 6.66)}{(31.59 + 6.66)} = 35.6\%$$

4.5.2 System Installation and Maintenance Costs

Installation Costs: the installation costs of two Jellyfish units are presented in the following table

Table 4-8
Installation Costs of Two Jellyfish™ Systems in Western Ave and Flagg Street

	# of units	Unit Price	Total
Hydroslide System (installed)	2	$43,000	$86,000
Jellyfish System (installed)	2	$85,000	$170,000
By pass 6'manholes (installed)	4	$16,000	$64,000
Pipe (LF installed)	200	$90	$18,000
Surface restoration (lump sum)	1	$80,000	$80,000
Other	1	$15,000	$15,000
Installation cost Western Ave	1		$216,500
Installation cost Flagg St.	1		$216,500
Total			$433,000

O&M Costs:

In order to calculate the annual number of cleanings and estimate the O&M costs, the following assumptions were made:

1. Each cartridge can remove 50 lb. of solids before being flushed
2. The annual average TS concentration in stormwater is 100 mg/L
3. The overall TS removal efficiency of the Jellyfish system is equal to 80%
4. Based on annual deflected flow and average TS concentration and removal efficiency, a total of 23 cleanings per year is required between the two units
5. Three staff members and one vactor truck are needed for cleaning
6. It takes four hours to clean one Jellyfish unit
7. Cartridges in the Jellyfish need to be replaced every third year

Annual maintenance cost:

Cost of each inspection = $4\text{hours} \times (3\text{ people} \times \$80/\text{h} + 1\text{ truck} \times \$100/\text{h}) + \$300/\text{truck (disposal)} \times 1\text{ truck}/2\text{ units} = \$1{,}510$

Annual cost = $\$1{,}510 \times 23\text{ cleanings} = \$34{,}730$

Cartridge replacement and disposal cost (every third year) = $= 2\text{ Jellyfish} \times 16\text{ cartrdiges} \times \$750\text{ installed cartridge} = \$24{,}000$

<u>Present Cost for the next 20 years:</u>

$$\text{Present Cost of Jellyfish} = \$433{,}000 + \sum_{i=0}^{20} \frac{\$34{,}730}{(1+0.02)^i} + \sum_{k=2}^{20} \frac{\$24{,}000}{(1+0.02)^k} = \$1{,}171{,}680$$

Where k refers to every third year.

$$\text{Present Cost for BMPs} + \text{Deflection to Jellyfish} = \$7{,}602{,}622 + \$1{,}171{,}680$$
$$= \$8{,}774{,}302$$

4.6 ALTERNATIVE 4: CONVENTIONAL BMP AND DEFLECTION TO A SORBTIVE™ FILTER

4.6.1 Estimated TP Removal

$\text{TP reduction with BMPs } (\%) = 6.18\%$ (see BMP section).

Calculations are based on the largest possible configuration of the Sorbtive Filter which can handle up to 1.65 cfs (8'x18' vault with 41 cartridges)

System performances were calculated assuming using two assumed TP removal efficiencies: 50% (conservative) and 70% (optimistic). Assumptions on TP removal efficiencies were described in Section 4-2. Results for different system settings, TP removal efficiencies, and Hydroslide® models are shown in the following tables.

Table 4-9
Flow Deflection and TP Removal Performances for Different System Settings Using a Constant Flow Hydroslide® and a 1.65-cfs Sorbtive™ Filter

Location	Spill elevation (ft)	Hydroslide max. deflection flow (cfs)	Annual deflected volume (MG)	% TP deflected	% TP removed (70% efficiency)	% TP removed (50% efficiency)
Western @ Jay Street	0	1.65	19.99	66.83	46.78	33.41
Western @ Jay Street	0.05	1.65	19.48	65.48	45.84	32.74
Western @ Jay Street	0.1	1.65	18.55	62.99	44.09	31.49
Western @ Jay Street	0.15	1.65	17.34	59.71	41.80	29.855
Western @ Jay Street	0.2	1.65	15.97	55.7	38.99	27.85
Western @ Jay Street	0.25	1.65	14.49	51.07	35.75	25.53
Western @ Jay Street	0.3	1.65	12.95	45.89	32.12	22.94
Flagg Street	0	1.65	5.7	71.7	50.19	35.85
Flagg Street	0.05	1.65	5.54	71.09	49.76	35.54
Flagg Street	0.1	1.65	5.25	69.95	48.96	34.97
Flagg Street	0.15	1.65	4.88	68.4	47.88	34.20
Flagg Street	0	1	4.88	48.15	33.705	24.07
Flagg Street	0.4	1.65	2.49	54.79	38.35	27.39

***Shaded rows indicate system configurations that guarantee the maximum possible TP reduction**

Table 4-10
Flow Deflection and TP Removal Performances for Different System Settings Using a Hydroslide® with Shutoff Followed by a 1.65-cfs Sorbtive™ Filter

Location	Spill elevation (ft)	Hydraulic Head (# of diameters) before shutoff	Hydroslide Max. Deflection Flow (cfs)	Annual Deflected Volume (MG)	% TP deflected	% TP removed (70% efficiency)	% TP removed (50% efficiency)
Western @ Jay Street	0	2	1.65	17.04	58.97	41.279	29.48
Western @ Jay Street	0	3	1.65	19.86	66.48	46.536	33.24
Western @ Jay Street	0.1	2	1.65	15.61	55.13	38.591	27.565
Western @ Jay Street	0.1	3	1.65	18.42	62.64	43.848	31.32
Western @ Jay Street	0.2	2	1.65	13.02	47.85	33.495	23.92
Western @ Jay Street	0.2	3	1.65	15.84	55.36	38.752	27.68
Flagg Street	0	2	1.65	5.23	58.03	40.621	29.01
Flagg Street	0	3	1.65	5.7	71.7	50.19	35.85
Flagg Street	0.05	2	1.65	5.07	57.41	40.187	28.70
Flagg Street	0.05	3	1.65	5.54	71.09	49.763	35.54
Flagg Street	0.1	2	1.65	4.79	56.28	39.396	28.14
Flagg Street	0.1	3	1.65	5.25	69.95	48.965	34.97

Therefore, the maximum possible TP removal with the optimistic and the conservative scenarios would be as follows:

1. Optimistic scenario (70% TP removal efficiency)

$$\text{Total TP removal (\%)} = 6.18 + \frac{(46.78 \times 31.59 + 50.19 \times 6.66)}{(31.59 + 6.66)} = 53.5\%$$

2. Conservative scenario (50% TP removal efficiency)

$$\text{Total TP removal (\%)} = 6.18 + \frac{(33.41 \times 31.59 + 35.85 \times 6.66)}{(31.59 + 6.66)} = 40.0\%$$

4.6.2 System Installation and Maintenance Costs

Installation Costs: the installation costs of two Sorbtive ™ Filter systems are presented in the following table

Table 4-11
Sorbtive™ Filter Purchase and Installation Costs

Equipment	Number	Unit Price	Total
Hydroslide System (installed)	2	$43,000	$86,000
Stormceptor (purchase)	2	$35,000	$70,000
Sorbtive Filter (purchase)	2	$115,000	$230,000
Installation of Sorbtive Filters and Stormceptor	2	$65,000	$130,000
By-pass 6' DMH (installed)	4	$16,000	$64,000
Pipe (LF installed)	200	$90	$18,000
Surface restoration (lump sum)	1	$80,000	$80,000
Other	1	$15,000	$15,000
Installation cost Western Ave	1		$346,500
Installation cost Flagg St.	1		$346,500
Total			$693,000

O&M Costs: in order to calculate the annual number of cleanings and estimate the O&M costs, the following assumptions were made:

1. Each cartridge can remove 100 lb. of solids before being replaced
2. The annual average TS concentration in stormwater is 100 mg/L
3. The overall TS removal efficiency of the Sorbtive Filter is equal to 80%
4. Based on the annual deflected flow, cartridge capacity, average TS concentrations and TS removal efficiency, a total of 4 cleanings per year is required between the 2 units
5. Three staff members and one vactor truck are needed for each cleaning
6. Two staff members are needed for routine inspections
7. It takes four hours to clean one filter unit and two hours to inspect it
8. Cartridges in the Sorbtive Filter need to be replaced every other year

Annual Maintenance Cost:

Cost of each routine inspections = $2\text{ staff} \times 2\text{ hours} \times \$80/\text{h} = \$320$

Annual cost of bi-monthly inspections = $\$320 \times 6 \times 2\text{ units} = \$3{,}840$

Cost of flushing the unit = $4\text{h} \times (3\text{ staff} \times \$80/\text{h} + 1\text{ truck} \times \$100/\text{h}) + \$300\text{ (disposal)} = \$1{,}660$

Annual cost for flushing = $\$1{,}660 \times 4$ times between the two units = \$6,640

Total annual O&M cost = \$3,840 + \$6,640 = \$10,480

Replacement Cost (every other year) :

Total cost of replacement (both sites) = $= 41 \text{ cartridges} \times 2 \text{ units} \times \$500 \text{ per cartridge (change and dispose)} = \$41{,}000$

<u>Present Cost for the next 20 years:</u>

$$\text{Present Cost of Sorbtive System} = \$693{,}000 + \sum_{i=0}^{20} \frac{\$10{,}480}{(1+0.02)^i} + \sum_{k=1}^{19} \frac{\$41{,}000}{(1+0.02)^k} = \$1{,}213{,}266$$

Where *k* refers to odd years

$$\text{Present Cost for BMPs} + \text{Deflection to Sorbtive System} = \$7{,}602{,}622 + \$1{,}213{,}266 = \$8{,}815{,}888$$

4.7 SUMMARY OF TP REMOVAL EFFICIENCIES AND COSTS

Table 4-12
Summary Table of Costs and TP Reductions for Individual Management Practices

	Construction Cost	Annual O&M Cost	Replacement/ Deflection Cost	Present Value of Costs (20 years)	TP Removal	O&M Level of Commitment
Street sweeping (monthly, dry-vacuum	n/a	$22,936	n/a (Expected service life > 20 years)	$397,972	4%	Medium
Rain gardens	$1,328,580	$66,429	n/a (Expected service life > 20 years)	$2,481,218	0.18%	Medium
BMP Catch basin	$1,912,500	$162,000	n/a (Expected service life > 20 years)	$4,723,732	2%	Medium
Deflection to MWRA	$150,000	$15,840	$28,014	$910,930	59%	Medium
Deflection to Jellyfish	427,000	$34,730	$24,000	$1,171,687	44% (optimum)	Very high
Deflection to Sorbtive Filter	$693,000	$10,480	$41,000	$1,213,366	47% (optimum)	High

Table 4-13
Summary Table of Costs and TP Reductions for Combinations of Alternatives

	Construction Cost	Annual O&M Cost	Replacement/ Deflection Cost	Present Value of Costs (20 years)	TP Removal	O&M Level of Commitment
Conventional BMPs	$3,241,080	$251,365	n/a	$7,602,922	6.18%	Medium
Conv. BMPs + Deflection to MWRA	$3,391,080	$267,205	$28,014	$8,513,852	65.2%	Medium
Conv. BMPs + Deflection to Jellyfish	$3,668,080	$285,085	$24,000	$8,774,609	50.2 (optimistic)	High
Conv. BMPs + Deflection to Sorbtive Filter	$3,934,080	$261,845	$41,000	$8,816,288	53.2 (optimistic)	High

Note: A 65.2% TP reduction can be achieved through increased deflection of stormwater flow to the MWRA system. Under this scenario, the 20 year present value of cost is in the order of a 10% increase to the individual Deflection to MWRA alternative cost in Table 4-12.

Section 5
References

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